

**DEVELOPMENT AND DEMONSTRATION OF SUSTAINABLE  
TRANSPORTATION CONTROL CONCEPTS:  
ACTUATED CONTROL OPTIMIZER DESIGN**

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Final Report**



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**by**

**Carroll J. Messer, Ph.D., P.E.  
Research Engineer  
Professor of Civil Engineering**

**and**


**Mohammad Anwar  
Graduate Research Assistant**

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**Southwest Region University Transportation Center  
Texas Transportation Institute  
Texas A&M University System  
College Station, Texas 77843-3135**

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16. Abstract  <p>This research developed a design concept for the first "smart" fully traffic actuated signal controller based on principles of hierarchically distributed, feedback control systems. Some examination of the performance sensitivity and feasibility of the overall design concept was conducted using the microscopic simulation program, TRAF-NETSIM sponsored by Federal Highway Administration. The design concept was reviewed by a local signal manufacturer and was highly recommended for further development and testing.</p> <p>A working prototype controller, having the features described in the research report, was therefore recommended by the researchers for development in future research working closely with industry and using the best available laboratory simulation tools.</p> <p>This research was conducted by Texas Transportation Institute, Texas A&amp;M University System, with funding provided by the U.S. Department of Transportation, University Transportation Centers Program to the Southwest Region University Transportation Center.</p>					
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## EXECUTIVE SUMMARY

Full-traffic actuated signal controllers have vehicle sensors located on each approach detecting the arrival of individual vehicles and providing green time on demand. Traffic actuated signal controller units are widely used throughout the United States, and their operation usually provides good service, estimated by the 1985 *Highway Capacity Manual* to be 15 percent better than pretimed control for the same intersection design and traffic conditions. However, as traffic volume increases, current traffic actuated control efficiency drops. Moreover, current traffic actuated signal controllers are basically well calibrated and functionally flexible stimulus-response switches, relying heavily on indirect feedback from user traffic (by cell phones in some cases) and some direct operating status feedback to the traffic engineer to improve traffic control and operations. Current controllers do not know what traffic flow state or control parameter set they are trying to achieve. They are provided only ranges of values within which to operate and calibrated switch mechanisms to apply. Only the engineers experience guides the initial selection process for a given design. Operational performance is always uncertain and depends on the skill of the traffic engineer in selecting the control parameters.

This research conducted by Texas Transportation Institute (TTI) sought to develop and examine the first “smart” fully traffic actuated signal control design concept. This smart controller would identify the optimal process state for existing conditions and provide a true feedback loop of control status and operations, thereby permitting optimal control to be attained. The controller’s design concept was developed, but true testing of the design concept in operation was considered beyond the scope of the resources available. However, extensive testing of existing traffic actuated control over a wide range of control parameters was conducted in a simulated traffic environment using Federal Highway Administration’s (FHWA) microscopic traffic simulation program TRAF-NETSIM.

The “smart” traffic-actuated control design concept is described in the following report. The design concept offers several appealing features, including the hierarchical functionality of

distributed feedback control systems operating in real-time. The feedback controller should be provided with the optimal control and flow parameters of the system to be achieved over a period of time, not just short-term switching points of individual phases proposed for some research systems currently being developed for FHWA. The proposed “smart” control design concept was presented and discussed on 11-21-97 with Naztec, Incorporated, a traffic signal control systems manufacturer located in Sugar Land, Texas. Quoting a written follow-up letter from the president of Naztec, “The ideas and theory are so unique that we at Naztec want to try some of the ideas in real controller applications. We are happy to be involved and feel that it is a privilege to continue to monitor your envisioned research program on the subject.”

Accordingly, it is recommended that prototype a smart traffic-actuated traffic signal controller be developed and tested within TTI’s hybrid traffic signal control/traffic simulation laboratory testbed to further extend and verify the attractive features offered by the new design concept developed within this study. The proposed research and development process should be conducted in concert with interested industrial partners.

## **DISCLAIMER**

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# CHAPTER 1

## INTRODUCTION

### PROBLEM STATEMENT

Traffic congestion is an increasing threat to the general quality of life in urban America. It is not clear that our current lifestyle is sustainable due to growing congestion in many areas, particularly those areas that are experiencing sizeable urban growth like Texas and many other urban environs. Texas urban traffic volumes have been increasing about 2-3 % per year (1), so the potential for increasing urban congestion looms even larger, particularly since most of our new second-generation urban freeways/toll roads have now been completed. Not only will the freeways become more congested, but the local arterial street system will also become more congested as motorists divert to the local street system to avoid the growing freeway congestion.

Traffic congestion is also a primary contributor to urban air quality problems. Vehicle emissions are much higher during congested stop/go traffic flow. At arterial traffic signals, vehicle emissions are highly concentrated and much more likely to produce health problems and exceed federal health and ambient air quality standards for carbon monoxide. Similar contributions to ozone air quality problems will also occur, particularly on days when atmospheric conditions are conducive to ozone alerts (a cool, clear, still, sinking air mass). A more sustainable lifestyle is enhanced by the provision of a more efficient traffic control system that reduces traffic congestion, resulting environmental impacts and energy consumption.

The efficient provision of mobility of persons and goods should directly consider the energy consumed, environmental impacts, and economic costs involved in providing transportation. This direct statement of operational objectives and resulting assessment of performance has not been provided by the traffic signal controller industry today in the United States. While microelectronics has been improved in traffic signal systems (as an example, the development of NEMA 1992 TS standards for signal system architecture and communications

design), the fundamental traffic control strategy used today remains locked in the operational concepts of the 1960's when explicit considerations of energy consumption and environmental impacts (e.g., on air quality) were nil.

“Modern” traffic signal controllers today are little more than calibrated electronic timers. Consider that controllers do not have a measurable traffic control objective. They also do not know how well the traffic is flowing, and they do not how to adjust their controls to move traffic more efficiently. Optimal traffic control can not be verified in the field with current controller designs. All operational enhancements rely on the ingenuity of the traffic engineer or technician, who is hopefully monitoring the traffic operations, to provide the optimizing adjustments.

This serious deficiency is a fundamental flaw in traffic control system specifications and design. It cannot be fixed by either increasing the speed of data collection or by buying faster computers, even though these improved features may provide increased performance. Recognizing and resolving this deficiency provides a major opportunity to develop an evolutionary traffic signal control system based on sustainable transportation principles given that real-time data acquisition systems are available. Texas A&M University is strategically located to work well with the traffic signal controller industry as two of the four largest traffic signal controller manufacturers (Eagle, Naztec) are located in Texas.

## **OBJECTIVES OF STUDY**

Research objectives define the focus of research that needs to be accomplished in an ordered fashion. This research project had the following three objectives of study to accomplish within a one-year time period:

1. Develop sustainable transportation control concepts that can optimally control traffic to minimize energy consumption, environmental impacts, and/or transportation costs. Real-time measurement is not now the critical operational issue; definition and subsequent optimization of sustainable traffic control objectives is, however.



2. Demonstrate the new control concepts using Federal Highway Administration's TRAF-NETSIM traffic simulation program and with TTI's TEXSim model, if needed.
3. Provide simple and relevant traffic control system design guidelines and specifications that would promote the manufacture of such an enhanced traffic control system by American signal manufacturers.

## **SCOPE OF RESEARCH**

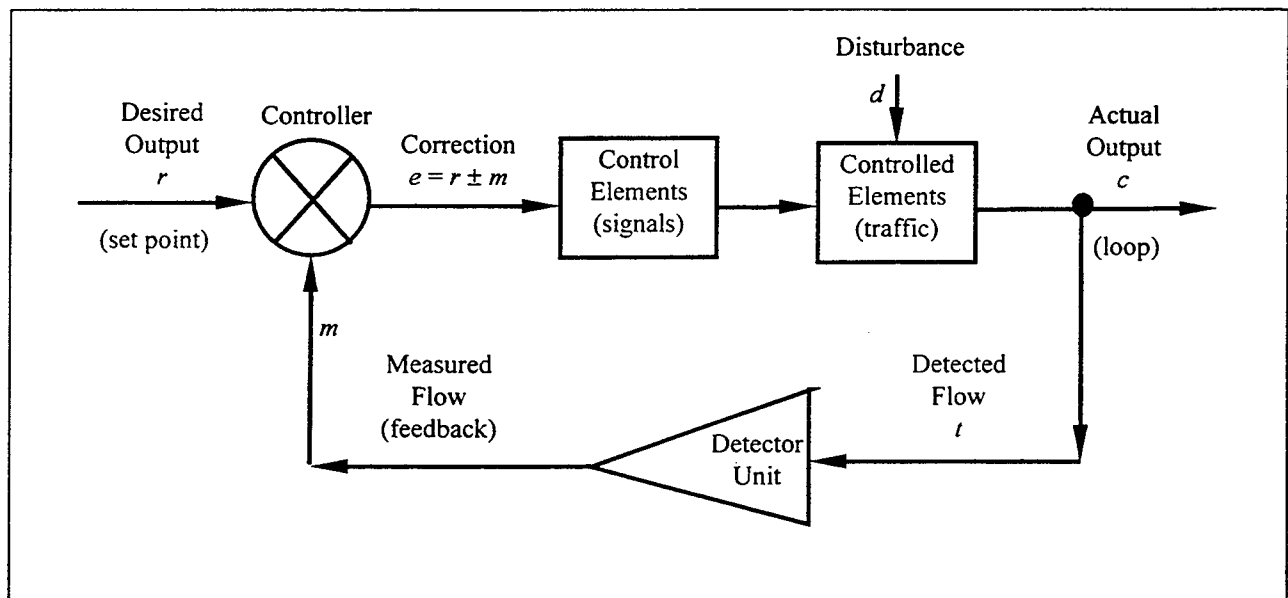
The scope of this research was limited to the consideration and development of an optimal control system design for traffic actuated signal control (NEMA TS 2) of traffic operations at an isolated intersection. Nominal inductive loop detectors were assumed to provide traffic data inputs to the controller. No arterial interconnection or coordination aspects, however important in some arterial cases, were considered herein. These arterial system coordination aspects could be considered in a subsequent research effort.

## **LITERATURE REVIEW**

A synthesis of the state-of-the-art of traffic control systems in the United States was recently published by Federal Highway Administration in 1996 (2). This massive document, the latest in a series of such publications since 1976, covers all types of traffic control systems, including traffic actuated signal control. The functions, features and guideline applications for traffic control systems, as currently applied in the profession, are clearly described in graphs, charts and tables contained in this handbook (2). Some of the basic definitions, functions, and features of isolated traffic actuated signal control to be described subsequently were drawn from this authoritative document. Traditional views and strategies prevail.

A basic reference which treats traffic controller design from a systems engineering perspective was written by Papageorgiou in the early 1980's (3). Hierarchical, adaptive feedback control systems was the fundamental architecture envisioned. This model follows a similar traffic control concept proposed earlier by Texas Transportation Institute in the late 1960's (4).

A basic local feedback control system is depicted in Figure 1. This local control system will be provided an important bit of information by the system —the desired process control state, or its targeted “set point” value. The set point value is the desired condition of the output stream following process control. The set point idea is analogous to the following room air-conditioning example. Suppose from human factors studies, others have learned that the optimal temperature of a worker’s office is 72°F for average summer weather and work conditions. The local office’s thermostat is then set at 72°F. The thermostat then proceeds to measure local room temperature in real-time. Should temperature rise above this set point value, the air-conditioning is turned on to reduce room temperature until such time as the local temperature is again measured to be equal to the set point value. Processed air flow only occurs when local measurement shows a need for it to achieve the desired temperature value. It is important to recognize that even this simple feedback controller knows its desired room temperature to be measured, the set point value of 72°F. This system framework will be used as the fundamental structure from which to identify the basic objectives and building blocks that must be identified and formulated into a working model of an optimal traffic actuated controller.



**Figure 1. Block Diagram of A Basic Continuous Feedback Control System(5).**

A more advanced feedback control system contains multiple levels of control features (such as local and system optimization) that can consider larger system perspectives, provide adaptive features based on local measurements and forecast of traffic and environmental conditions, and even learn from its previous experiences. The most advanced, or evolutionary systems, can even self-organize their control objectives, measurements, and functions into more efficient configurations based on self-diagnostic performance assessments and recognized environmental changes. A model of such an advanced hierarchical, adaptive, feedback control system is depicted in Figure 2 as developed for early freeway control systems work by TTI (4).

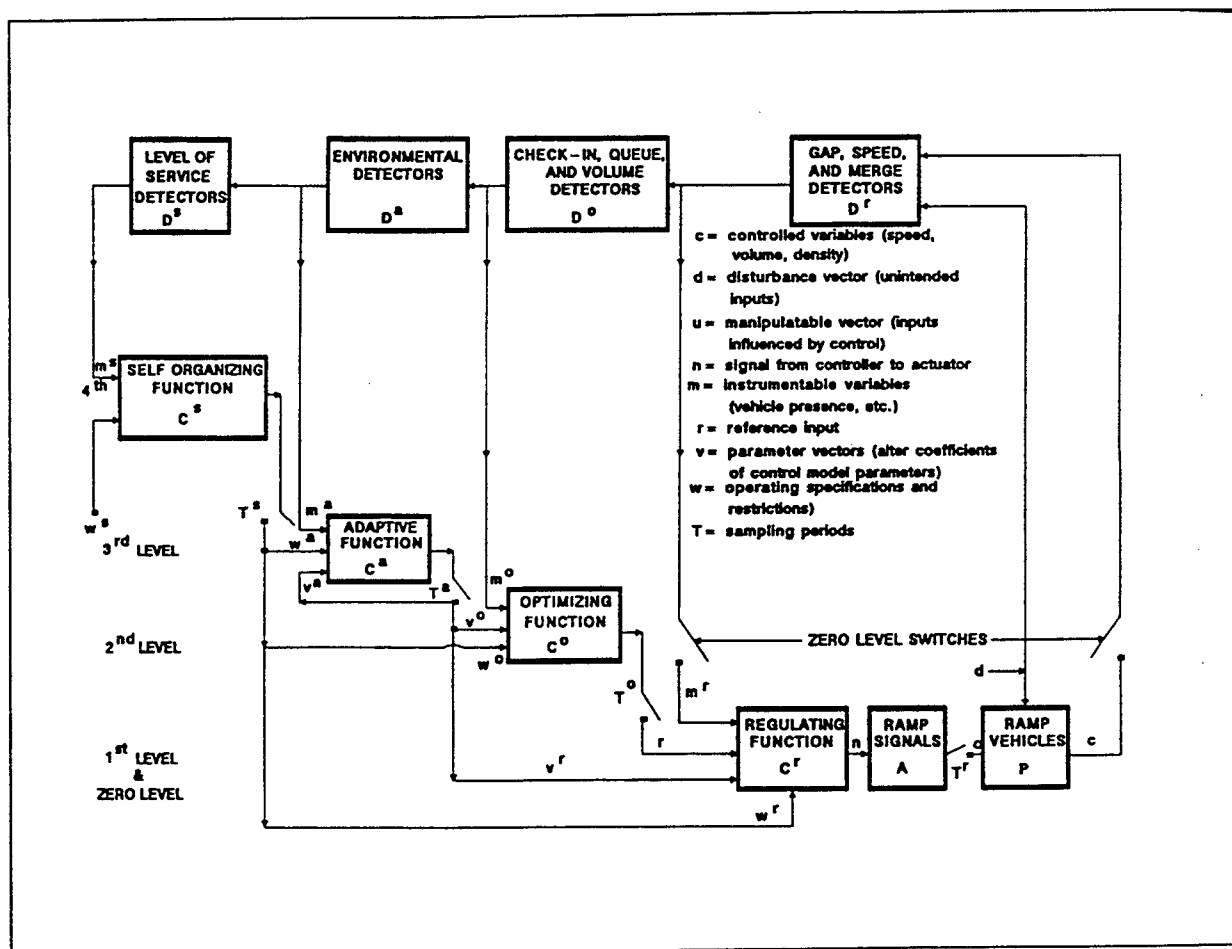


Figure 2. Hierarchical, Multilevel, Adaptive, Distributed, Feedback Control System (4).

An important feature of hierarchically distributed, feedback control systems is the notion that the adaptive optimal system control objective, or set point value in Figure 1, will be provided by the systems manager to the local controller. This value is currently “optimal” from a systemwide viewpoint, all things considered, but the value may change with conditions. This feature is analogous to the following additional air-conditioning example. Suppose from human factors studies, the system has learned that the optimal temperature of a worker’s office is 72°F for average summer conditions. However, the system determines that today will likely be unusually hot and, therefore, the system automatically adjusts the setting of the thermostat of the office to 74°F to optimally balance out cooling with electrical efficiency, given that a human factors model says that relative outdoor-to-indoor temperatures on an unusually hot day feel just as cool for one day as do nominal temperatures on an average day. Should the next day also be unusually hot, the temperature might be reduced to 73°F following some human adaptation rule.

Local traffic actuated signal controllers have no such optimal process control objectives expressed in measurable traffic flow terms. They do have defined critical gap times in the traffic stream such that when a gap greater than the set gap is measured, the service phase ends, but the controller does not know if the set gap is a good gap, nor how efficiently traffic is flowing.

## **REPORT ORGANIZATION**

The following chapters present the overall research methodology, experimental results, and findings. Chapter 2 presents the salient features of traffic actuated control for isolated conditions. Control systems engineering terminology are related to traffic control operations. Chapter 3 presents the experimental design followed in this research. The experimentation and measurement of traffic control performance were conducted using a nationally recognized microscopic traffic simulation program, TRAF-NETSIM, developed and funded by Federal Highway Administration (6). Using a valid simulation model expedited control of variables and assessment of results. Chapter 4 presents the results of the simulation experiments and provides a basis for the conclusions and systems design recommendations given in Chapter 5.

## **CHAPTER 2**

### **TRAFFIC CONTROL CONCEPTS**

#### **TYPES OF CONTROL**

Traffic controllers are used to operate the traffic signals at intersections that assign the right-of-way to conflicting traffic movements. The assigning of right-of-way using green and red signals occurs over repeated cycles of time. The traditional traffic controller unit is primarily a clock timing and electric power switching device. The National Electrical Manufacturers Association (NEMA) and various governmental organizations provide guidelines and standards for manufacturer, meaning, and operation of traffic signal systems in the United States. There are several ways to describe and/or classify the features and functions of traffic controllers. The following sections briefly summarize the main aspects of signal controllers.

#### **System Control**

Statistical studies show that there are about 1.0 traffic signals per 1,000 people living in an urbanized area of America. Thus, a small city, having a population of 100,000 people, would have about 100 traffic signals operating in it. Of these 100 signals, some will be located close together along arterial streets and downtown grid networks and be operated together in coordinated signal systems. Here the output traffic flows of a signal become the identifiable system inputs to a downstream signal.

Once signal spacing exceeds about 2 minutes travel time for arterial vehicular traffic (about 2 km) upstream, platooned output traffic flows have dispersed into practically random arrival flow patterns from which the traffic can no longer be identified as having any predictable short-term flow patterns. When all approaches to a signalized intersection have random flow (Poisson), and when its signal performance in no way depends on the performance of any other signal, then the intersection is said to be an “isolated intersection.” At an isolated intersection, the resulting traffic performance depends solely on the traffic, geometry and control timings of the subject signalized intersection.

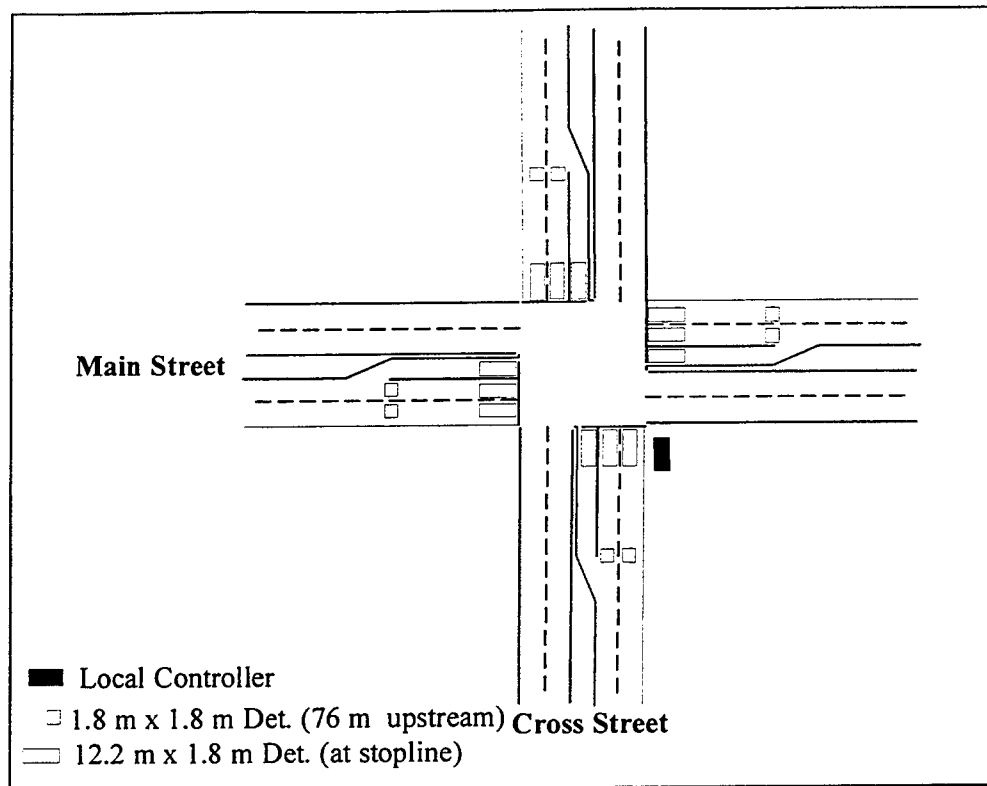
## **Local Controller Unit**

Traffic signal controllers (timers) can be pretimed or actuated. That is, their signal sequence and display are either prescheduled and predetermined, in the former case (pretimed), or determined in real-time by the measured presence (actuation) of arriving and/or standing vehicles at the intersection, in the latter case (traffic actuated). Microcomputer technology may be used in either case to time the lights or may even be used to count link traffic volumes. Some pretimed systems use system computers to measure traffic volumes, perhaps for 15 minutes, and then determine the best available pretimed timing plan to serve the current traffic pattern.

Traffic actuated controllers do not have a pretimed cycle, signal sequence, or green signal displays. They provide, extend and terminate signal green displays on the fly within lower and upper timing bounds and without any specific knowledge of existing traffic volumes or traffic control objectives (i.e., minus the set point values in Figure 1). Even today, a traffic actuated controller is simply a calibrated-stimulus-response switching mechanism (noted as Level 0 in Figure 2). It responds to detected inputs (vehicles arriving/stopped on sensor loops buried in the pavement) and continues to serve the traffic demand until a service breaking gap is detected (or maximum display time is reached) wherein it then switches to the next phase having call for service, at which time the process is repeated.

## **Signalized Intersection**

A typical high-type signalized intersection having full-actuated signal control is depicted in Figure 3. The major east-west street (Main) intersects the major north-south street (Cross). Each street is assumed to have two through lanes inbound on each approach and two departure lanes. Moreover, single-lane left turn lanes are provided to remove the left turns from impeding through traffic. The left turn traffic is provided a separate left turn signal phase that holds up opposing through traffic while the left turns are provided a protected left turn (arrow) phase. Separate auxiliary right-turn lanes are not provided in Figure 3. This example intersection (Figure 3) serves as the experimental testbed throughout this report. All experimental results to follow in Chapter 3 were developed from this local intersection design.



**Figure 3. Study Intersection and Traffic Actuated Detection System.**

## **ACTUATED CONTROL PARAMETERS**

The duration of the green in a basic traffic-actuated signal phase (green+yellow+red clearance) is affected by three timing parameters in addition to the design of the loop sensor, for a given traffic volume level per lane. These three timing parameters are described below. All are currently determined and set by the traffic engineer outside of the signal control process.

### **Minimum Green**

The Minimum Green is the shortest time that the related green signal will be displayed given a single call for service. Additional calls received during the Minimum Green will not extend the displayed green unless they either still exist at the end of Minimum Green, or their following gaps have not “gapped out” based on the timing of the Passage Gap.

## **Passage Gap**

The Passage Gap timing is the elapsed time during green since any service detector for the phase green has dropped its call (lost measured presence of vehicle in loop). For a single detector serving a phase green, the Passage Gap timing is the elapsed time after a vehicle has departed the loop detector out in the street, assuming that the loop is in the “presence mode” of operation. Complications arise when more than one loop is detecting traffic per lane, or one loop is detecting multiple lanes of traffic. In the former case, more detections occur than vehicle traffic volumes would suggest; in the latter case, fewer detections may occur than vehicle traffic volumes would suggest (less than one detection per vehicle passage). The Passage Gap is sometimes denoted as being the “Extension” or “Gap” time parameter by some traffic engineers.

The Passage Gap timing feature would be expected to extend the green signal display beyond the Minimum Green in proportion to the overall traffic volume level being detected on a related phase during green. The higher the traffic volume being detected during green, the longer the actual green extension expected beyond Minimum Green, or loss of presence detection.

The traffic signal design (primarily the location of any stopline detectors) will modify the initiation of Passage Gap timing, or what is known as the green signal phase trying to “gap out.” Gap Out timing does not begin until a loss of vehicle presence detection occurs (over the loops buried in the pavement) on the approach. Thus, when stopline detection is provided within a signal design, green extension timing will not begin until the queue, which has accumulated during red, is cleared from the stopline following initiation of the green signal. This initial green time required to clear the stopline queue is timed independently of the actual Minimum Green timing within the controller unit. Thus, for any given green phase, the beginning of the green extension timing occurs when the queue clears all detectors, but it cannot terminate until the Minimum (assured) Green expires. Theoretical models, described later, have been proposed for estimating the expected (or average) extension time of a green for a given approach volume and traffic condition, but most of these models assume that the extension detectors operate in the “pulse” mode of operation, and not in the typical presence mode.



## Maximum Green

Traffic actuated controller units provide a timing feature that limits the maximum green signal display that can be shown even when a constant call (constant traffic demand) for service exists. This limit sometimes occurs when one phase (approach movement) has a very high traffic load that exceeds the capacity of the signal to service it. Infrequently, a constant call may also occur because of electronic/mechanical failure of some component of the vehicle detection system.

## Signal Phase

Thus, the range of signal timings for an individual signal green display is between Minimum Green and Maximum Green, assuming that the phase is active (on). This range assumes that pedestrian signal overrides are not active for the phase. The pedestrian override may also extend a phase to satisfy pedestrian crosswalk operations. Thus, for a given phase

$$MinG \leq G(t) = G_p(t) + E_x(t) \leq MaxG \quad (1)$$

where:

$MinG$	=	selected minimum green timing parameter, sec;
$G(t)$	=	duration of green display for actuated phase, sec;
$G_p(t)$	=	duration of green until queued platoon clears, sec;
$E_x(t)$	=	extension of green display beyond minimum green, sec.; and
$MaxG$	=	selected maximum green timing parameter, sec.

An actuated signal phase is composed of three signal displays and timing intervals. These displays are the green, yellow and red clearance signal displays. During the timing of these three intervals, no other conflicting traffic movements will be shown a green signal (be given the right-of-way). Thus, a signal phase is composed of

$$\phi(t) = G(t) + Y + R_c \quad (2)$$

where:

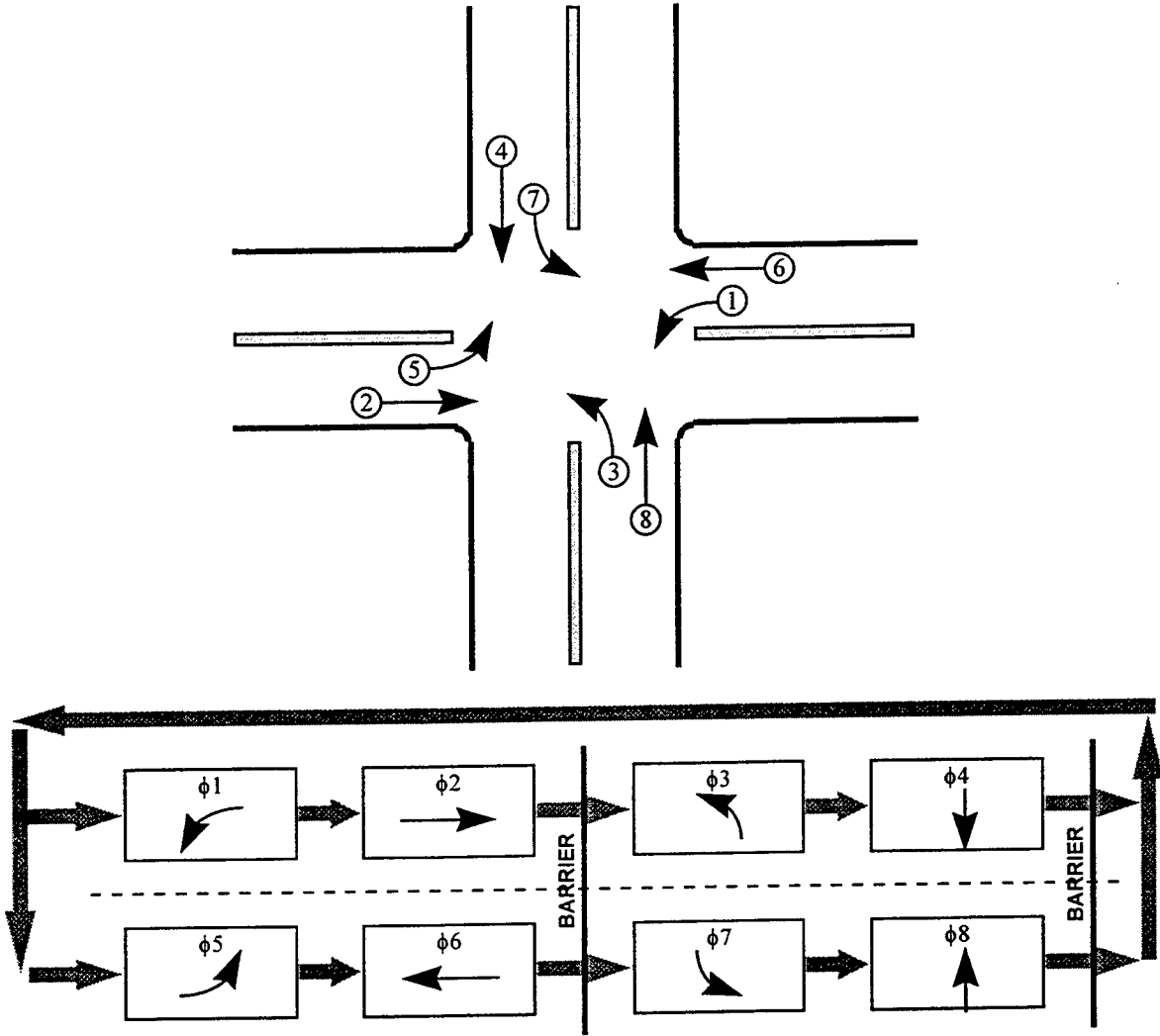
$\phi(t)$	=	total duration of a signal phase, sec;
$G(t)$	=	duration of green display, sec;
$Y$	=	pretimed duration of yellow warning interval, sec; and
$R_c$	=	pretimed duration of red clearance interval, sec.

In traffic actuated control, the duration of the green varies with each cycle of signal displays, but the duration of the yellow and red clearance intervals are preset for each phase and do not change over time unless changed by the traffic engineer (or timing plan).

## **SIGNALIZED INTERSECTION CAPACITY ANALYSIS**

The traffic engineer provides a sequence of signal phases within the signal controller unit that repeats itself every cycle when all phases have (detected) traffic present to serve. More than one phase can be present at a time, but no phase can be displayed simultaneously with another conflicting phase. In this research, the common eight-phase, quad-left leading phase sequence was chosen. This quad-left leading phase sequence is shown in Figure 4, using the National Electrical Manufacturers Association (NEMA) phase numbering scheme, which is also used in PASSER II. A dual-ring controller unit is assumed which operates conflicting pairs of movements for a street in the same ring.

The cycle duration varies from cycle to cycle in actuated control. It is determined by the time required to serve (and then safely and efficiently) gap out the controlling phase pair for each street. As an example, Phases 1 and 2 may be controlling Main Street (East-West) and Phases 3 and 4 may be controlling Cross Street (North-South). Obviously, Phases 5 and 6 together with Phases 7 and 8 could likewise have been the controlling phases. Note, the controlling phases may vary on a cycle-by-cycle bases, but average controlling phases are usually determined for a given time frame when conducting signal timing and capacity analyses.



**Figure 4. Eight-Phase, Dual-Ring, Quad-Left, NEMA Signal Phasing.**

Thus, for average conditions, the average cycle time of a signal sequence can be thought of as being composed of the following four critical phases for the high-type intersection in Figure 4, having eight major phases (with no overlaps shown) in two concurrently timing rings:

$$C = \phi_{c1} + \phi_{c2} + \phi_{c3} + \phi_{c4} \quad (3)$$

The argument subscript  $c$  indicates that the phase is a controlling (or critical) phase for the cycle, and the number in sequence represents the current step that the controlling phase is in the four-phase sequence. The durations of the critical signal phases' displays over a cycle would be

$$(G + Y + R_c)_{c1} + (G + Y + R_c)_{c2} + (G + Y + R_c)_{c3} + (G + Y + R_c)_{c4} = C \quad (4)$$

Let a phase,  $\phi$ , also be defined in terms of effective green ( $g$ ) and lost ( $l$ ) times, based on signal capacity considerations, as

$$\Phi = G + Y + R_c = g + l \quad (5)$$

where the effective green is the time saturation flow occurs at the stopline, and the lost time per phase, due to starting and stopping platoons, is about four seconds per phase, or

$$(g + l)_{c1} + (g + l)_{c2} + (g + l)_{c3} + (g + l)_{c4} = C \quad (6)$$

Thus, the sum of the four critical phases' effective green available for saturation flow at the intersection is equal to the total effective green time available, or

$$g_{c1} + g_{c2} + g_{c3} + g_{c4} = C - n l = C - L \quad (7)$$

### Phase Capacity Analysis

The total traffic demand upon the intersection geometry and signal controller affects the resulting traffic operations that can be provided. The *Highway Capacity Manual* (7) of the Transportation Research Board provides the national methodology for analyzing the capacity and level of service provided by an isolated signalized intersection, pretimed or actuated. The fundamental capacity analysis methodology can be developed from the above concepts and the following basic notions. A signal phase that provides an "effective green time"  $g$  for moving traffic into the intersection (across the stopline) at saturation flow rate  $s$  (vphg) has an hourly capacity  $c$  (vph) when the approach is effectively green for  $g/C$  fraction of the hour.

Mathematically, this statement of hourly phase capacity is equivalent to

$$c = \frac{g}{C} S \quad (8)$$

where:

- $c$  = phase capacity per hour, vph;
- $g$  = effective phase green time, sec;
- $C$  = cycle time, sec; and
- $S$  = saturation flow rate, vphg.

Operational analysis of signalized intersections is normally based on the assumption of “average pretimed” conditions, even for actuated signals. A considerable amount of research has been conducted recently (8) which has investigated many of these factors. For this research, it is assumed that effective green time is about equal in duration to the displayed green time, and that the saturation flow is assumed unimpeded by all other flows and turning movements.

### Phase Degree of Saturation

When a signal phase is servicing its full phase capacity, it is said to be “saturated” or its volume-to-capacity ratio,  $X_\phi$ , is said to be 1.0. Oversaturated phases, have more traffic volume,  $V$ , trying to use the phase than the phase has capacity to serve it. Stopline queuing and delays increase dramatically when the degree of saturation,  $X_\phi$ , exceeds 1.0. A phase’s degree of saturation (v/c ratio) is calculated from terms defined above as

$$X_\phi = \frac{V}{c} = \frac{V}{\frac{g}{C} S} = \frac{V C}{g S} \quad (9)$$

Intersection analysis to follow solves for effective green,  $g$ , in Equation 9 in terms of the remaining four variables.

### Intersection Capacity

A capacity analysis of the total intersection operations of Figure 4 can now be conducted using Equations 7 and 9, especially for the critical controlling phases (and related traffic movements). Substituting the phase capacity results (Equation 9) for the individual phase durations of the total cycle time equation (Equation 7) yields the following equation in terms of traffic, geometry and signal cycle time. The resulting equation is

$$\left(\frac{VC}{XS}\right)_{c1} + \left(\frac{VC}{XS}\right)_{c2} + \left(\frac{VC}{XS}\right)_{c3} + \left(\frac{VC}{XS}\right)_{c4} = C - L \quad (10)$$

The cycle,  $C$ , is then moved to the right-hand side of Equation 10. Signals should be timed so that all the critical signal phases provide the same degree of saturation,  $X$ . Assuming this is done, the sum of the critical flow ratios at the intersection,  $y_{ci} = V/S_{ci}$ , then equals

$$Y = \left(\frac{V}{S}\right)_{c1} + \left(\frac{V}{S}\right)_{c2} + \left(\frac{V}{S}\right)_{c3} + \left(\frac{V}{S}\right)_{c4} = \frac{X(C-L)}{C} \quad (11)$$

for a given cycle  $C$ . Thus, the higher the volumes, the higher the intersection's flow ratio,  $Y$ . The flow ratio,  $Y_{X=1}$  that results in the intersection being saturated for a given cycle, can be found by setting  $X=1$  in Equation 11, and solving for  $Y$  for a given cycle,  $C$ , while assuming a constant total intersection lost time of about 16 seconds.

This critical cycle, which results in the intersection being at capacity on its critical phases,  $C_{X=1}$ , can be found from Equation 11 for a given  $Y$  for  $X=1$  as

$$C_{X=1} = \frac{L}{1-Y} \quad (12)$$

where:

- $C_{X=1}$  = saturation level cycle time, sec;
- $L$  = total of  $n$  ( $4 \times 4.0 = 16.0$  sec) critical phases lost times, sec; and
- $Y$  = total of  $n$  critical phases critical flow ratios, sec.

If uniform (not random) flow generates a total intersection demand flow ratio  $Y$  using the intersection operating at the cycle,  $C_{X=1}$ , then all critical arrival queues stopped at the intersection during red would just clear the intersection as each critical phase terminates when timed to provide equal degrees of saturation. No queues would overflow into the next cycle if all arrival flows were uniform and at capacity. Since arrival flows are not uniform, numerous cycles will needlessly overflow unless the average cycle is increased above the minimum cycle given in Equation 12 for uniform flow and lane distribution.

### Minimum Delay Cycle

Traffic engineers seek to operate the traffic signals to provide “optimal” operations for a given volume ( $V$ ) and geometrics ( $S$ ). Optimization criteria may include minimizing delay, stops, fuel consumption (a linear combination of delay and stops), or total operating cost. Traffic actuated control may have different optimal operating parameters than pretimed control. Earlier research (circa 1976) by Courage (9) found that traffic actuated control, using small-area (point) detection, had a minimum delay cycle that was thirty percent (30%) larger than the saturation cycle. Courage’s minimum delay cycle for actuated control thus could be calculated from (2, 9):

$$C_{oA} = \frac{1.3 L}{1 - Y} \quad (13)$$

Minimum fuel consumption and minimum stop rate cycle times were respectively about ten and twenty percent longer than given by the Equation 13 minimum delay cycle for actuated control.

One of the most famous equations in traffic engineering is Webster’s minimum delay cycle length equation developed for pretimed control, which was developed from early computer simulation and theoretical studies conducted in the late 1950’s. Webster’s minimum delay cycle length equation for pretimed control is determined from (10):

$$C_{oP} = \frac{1.5 L + 5}{1 - Y} \quad (14)$$

Inspection of the above cycle equations reveals that the minimum delay cycle for pretimed control is about 80 % longer than the saturation cycle and about 40 % longer than Courage's minimum delay cycle for actuated control. The ability of a traffic actuated controller to change its green splits as arrival volumes change on a cycle-by-cycle basis theoretically should permit actuated controllers to operate more efficiently at a lower cycle for a given average volume. Actuated control, when operating at a slightly lower cycle than pretimed control, should therefore produce lower delay. Thus, a lower minimum delay cycle should be expected for actuated control than for pretimed control, as given by Equations 13 and 14 above.

## DESIGN PROBLEM

Let it be assumed at this point that an optimal (minimum) delay cycle length exists for actuated control of the form estimated by Equation 13. If so, then  $C_o$  could be used as the local optimal target or "set-point" value to direct the operations of the local controller for a period of time. A real-world problem would still remain in applying this hypothesis within NEMA controllers. How would the controller count the traffic volumes,  $V$ , in Equation 13, assuming that the phase saturation flow rates,  $S$ , could be estimated exogenously for all approaches?

A straightforward analytical method exists for estimating the critical flow ratio,  $y_p$ , for a signal phase instead of having to estimate the arrival volume,  $V$ , and saturation flow,  $S$ . Automatically measuring traffic volumes in the field, even using ITS-level video technology, is difficult to do accurately for a variety of reasons, although measuring the presence of vehicles at the stopline is not, as this is routinely done by delayed-call presence detectors located at the stopline. The flow ratio loading an individual signal phase can be analytically estimated from queuing theory (using the input-output model) from measurements made for the phase at the stopline from

$$y = \frac{V}{S} \approx \frac{g_p}{r + g_p} \quad (15)$$



where:

$y$	=	flow ratio of a (critical) approach;
$V$	=	average arrival volume (demand) on the approach, vph;
$S$	=	saturation flow at stopline on the approach, vphg;
$g_p$	=	measured duration of platoon clearance time at the stopline, sec; and
$r$	=	measured duration of previous red display preceding measured green, sec.

### OPTIMAL SET POINT

Determination of the optimal “set point” cycle that the actuated controller should provide is straightforward. Logic would be provided in the controller to find the critical phase sequence and resulting intersection flow ratio (Equation 11) from which the optimal (minimum delay) cycle length would be determined (Equation 13). Perhaps as many as 25 cycles might be monitored, traffic performance measured, or estimated per cycle, and results averaged. The duration of the averaging time (number of cycles) would be selectable, but it probably would be determined by field testing. The beneficial feature of this method is how simply the optimal cycle can be determined from direct time measurements made from stopline detectors which are often provided in the original signal design. NEMA controllers already measure detection and phase times accurately.

At least two research questions remain. Do traffic actuated controllers really have an optimal (minimum) delay cycle, as some research publications suggest for some designs, and, if so, should average or critical lane volumes be used to estimate the minimum delay cycle? The following chapter presents our NETSIM simulation experiments conducted to help answer these and other questions.



## **CHAPTER 3**

### **EXPERIMENTAL DESIGN**

#### **OVERVIEW**

This chapter deals with the experimental design procedures employed to accomplish the objectives of this research effort and to answer the two research questions noted at the end of Chapter 2. Computer simulation was used extensively in this regard. In this chapter, the testbed used for the study of traffic actuated intersections is presented along with the designs for studying lane utilization, occupancy, and performance measures under varied geometric, signalization, and traffic conditions. The following sections deal with the study design, statistical framework, and analysis methodology employed in this study.

#### **RESEARCH PROTOCOL**

A microscopic traffic simulation model was exercised on a hypothetical intersection testbed to examine vehicle performance while operating on a wide variety of traffic actuated signal timing strategies. The detailed research program is described below.

##### **Simulation Model**

The TRAF-NETSIM simulation model (6) was used to study isolated signalized intersections under fully actuated control. NETSIM was chosen not only because it can simulate signalized intersections under fully-actuated control, but also for its graphics capabilities. NETSIM has been developed under the long sponsorship of Federal Highway Administration.

TRAF-NETSIM is a microscopic traffic simulation model that uses a series of queue-discharge, car-following, lane switching and probability algorithms to simulate traffic flow on an urban street network, typically in one-second time slices. The street network is represented in terms of a series of interconnected links and nodes that are divided into a set of uni-directional links and nodes. The program simulates individual vehicle trajectories as they are emitted into and move through the network.

NETSIM requires detailed information to be input for modeling signalized intersections. Approach volumes by movement, approach speeds, geometric inputs like number of lanes, and lane designations must be coded. For actuated intersections, additional information is required like the minimum and maximum green time, vehicle extension interval, duration of yellow and red clearance time for each phase, and detector data including location, type and size of detector.

NETSIM outputs include a range of traffic performance measures. Some of the relevant performance measures that are generated are average total delay, percentage stops and total fuel consumption. Also, NETSIM can provide the user with a second-by-second account of detector occupancy, vehicle count, and signal indication.

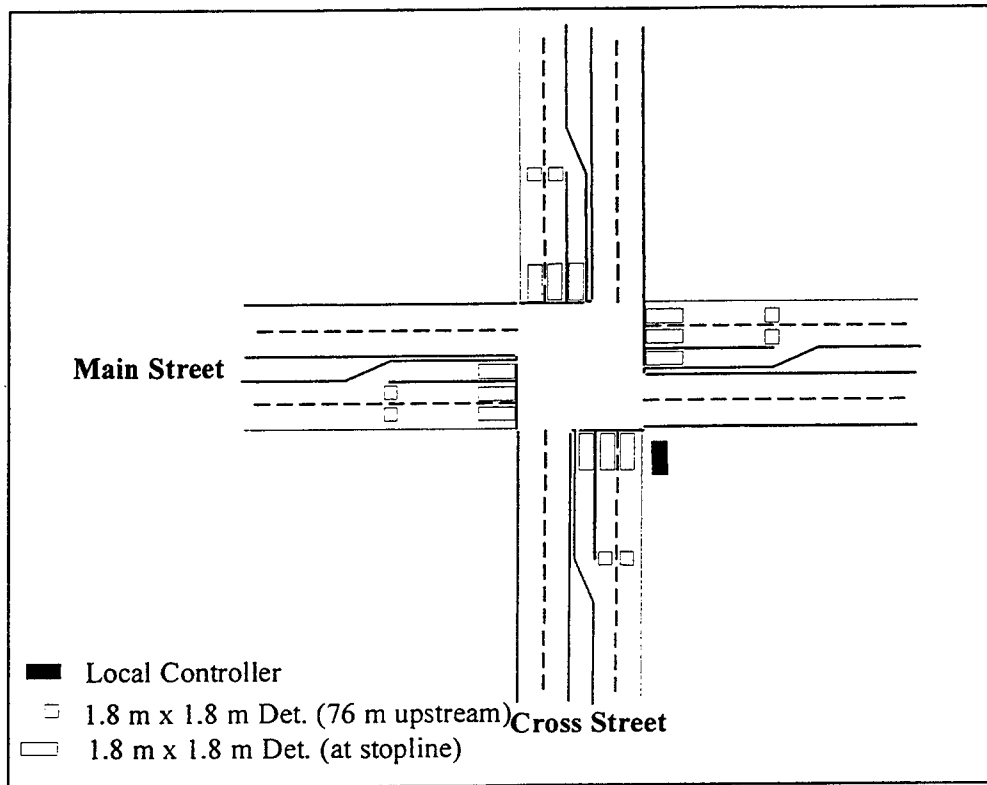
### **Traffic Actuated Intersection Testbed**

A four-leg isolated intersection under fully-actuated control was simulated in TRAF-NETSIM. The isolated intersection study testbed is presented in Figure 5. The base design and the range of conditions simulated for each study are presented in the following sections. NETSIM simulations were used to obtain estimates for lane utilization, detector occupancy and performance measures like average intersection delay per vehicle, percentage stops and fuel consumption for a variety of conditions.

The base intersection geometry had two lanes for through traffic with the curb lane shared with right turning traffic and a bay for left turning traffic. The link length on each approach was 183 meters and the length of the bay was 91 meters. The saturation flow rate was assumed to be 1800 vphg per lane and the start-up lost time coded was 2.5 seconds. The assumed speed on the links was 64 km/hr. The actuated controller was coded to simulate an 8-phase NEMA controller with the left turning phase leading the through phase along with dual-ring overlaps. Right turns on red were not allowed on any of the approaches.

Stopline detectors were coded on all lanes and were 12.2 meters long. These detectors turn off when the initial queue clears the stop line. In other scenarios where a minimum green

time was coded for the through and left-turning phases, the stopline detectors were used as standard calling detectors and did not extend the green for the phase. On the through lanes, upstream detectors were 1.8 meters square and located 76 meters from the stopline.



**Figure 5. Experimental Testbed for Signalized Intersection with Full-actuated Control.**

The maximum green time for the through phases was set at 41 seconds and the maximum green time for the left-turning phase was set at 16 seconds. The yellow interval was 3 seconds and the all red interval was 1 second for each signal phase. The main street through phases 2 and 6 (NEMA phasing scheme) were on minimum recall. Traffic arriving at the intersection split into 15% left turning, 70% through and 15% right turning on each of the four approaches. Arrival volumes considered were between 400 and 1500 vph on each approach.

## LANE UTILIZATION STUDY

Lane utilization describes how traffic is distributed across available lanes. On a single lane approach, the arriving traffic on that approach has to use that lane. The lane utilization factor for that lane is 1.0. On a two-lane approach, on the average one can expect that the number of vehicles using each lane is half of the total arriving volume on that approach. However, during any given cycle due to the randomness in arriving traffic, one lane may be used by more vehicles than the other. Hence, the lane utilization factor for the lane group is greater than 1.0. Lane usage may be uneven due to the amount of left and right turning traffic at the intersection or even at an intersection downstream of the study intersection. Drivers tend to shift to the curb lane when they expect to make a right turn at the next intersection, thereby creating a difference in the usage of the curb lane as compared to the interior lanes.

Lane utilization factor is estimated herein as an average of the ratio of the maximum lane volume per cycle over the average lane volume per cycle. The average is taken over many cycles for the same traffic conditions. The lane utilization factor is important especially at actuated intersections with stopline detectors that not only call a phase but also extend the phase until the queue clears. The imbalance in lane usage allows the phase to extend longer than it normally would under average traffic conditions, since a longer queue on a lane takes a longer time to clear. The length of the saturated portion of the green is estimated as the length of time the stopline detector is occupied after the phase turns green until the detector is no longer occupied once the queue is discharged. The saturated portion of green can then be used to estimate the volume on the approach.

The next section of this report deals with the study to estimate arrival volumes and minimum delay cycle length based on estimated critical flow ratios and lost time per cycle. The lane utilization factor can be used to convert the estimated volumes on the critical lane to average volumes on the approach. Hence, it is important to obtain reliable estimates of the lane utilization factor.

The TRAF-NETSIM traffic simulation software was used to simulate an isolated actuated intersection. The base design, as mentioned in the previous section, was used along with some changes to study lane utilization in NETSIM. The minimum green time was set at 1 second (minimum allowed) and the stopline detectors were coded such that they allowed a phase to extend until reaching the maximum green time or as long as there was a queue discharging on the corresponding lane on the approach. Two different vehicle extension values of 1.1 seconds (minimum allowed) and 5.0 seconds were coded. The arrival volumes were coded to be the same on three approaches and half on the fourth approach. An alternate geometric scenario was considered in which the number of through lanes was changed to three lanes on each of the four approaches to the intersection.

Surveillance detectors (a NETSIM feature) were coded on each through lane at the stop line to determine the vehicle counts on a second by second basis. The vehicle counts per cycle per lane could be used to determine the lane utilization factor. The NETSIM program was also coded to obtain an output of the signal indication on a second by second basis, which could be reduced to NETSIM generated green times and cycle lengths by movement for the analysis time period. Runs were replicated four times with different random seed numbers to get 40-80 cycles for each condition, and the results obtained were averaged over them.

FORTTRAN programs were written to process the output to obtain cycle length and vehicle counts by lane per cycle. Once the data were generated using TRAF-NETSIM, exponential models were calibrated using statistical means. The calibrated parameter values were obtained using numerical search methods to minimize the squared error between the NETSIM observed values and model exponential estimated values. The exponential model was calibrated for lane utilization as a function of the average arrivals per cycle for each lane configuration. The model form is as follows:

$$LU = A + (n - A) e^{Bm} \quad (16)$$

where:

- $LU$  = lane utilization;
- $m$  = average arrivals per cycle;
- $n$  = number of lanes; and
- $A, B$  = calibration parameters for each lane configuration.

The lane utilization factor was estimated by averaging the ratio of the maximum lane volume per cycle over the average lane volume per cycle for the entire analysis period and over different NETSIM replications. This lane utilization factor was used in the calibration of the parameters  $A$  and  $B$  in the above model.

## OCCUPANCY AND GREEN TIME STUDY

The purpose of this study is to obtain estimates of the volume on an approach from which the resulting flow ratio can be estimated. The estimated flow ratios can then be used to determine a minimum delay cycle length for a particular scenario. Volume and flow ratio can then be determined from the occupancy of stopline detectors. The length of time the detector is occupied after the start of green until the first time all of the detectors are no longer occupied (indicating that the queue has cleared) is called “the saturated portion” of the green time. Knowing the saturated portion of green, the cycle length and the effective red time, it is possible to estimate the flow ratio for the lane group.

Flow ratio is defined as the arrival demand over the saturation flow rate for the approach lane group. From the flow ratio, the arrival volumes can be estimated by multiplying the flow ratio by the saturation flow rate for the lane group. As noted in Chapter 2, the lane group flow ratio for the  $i$  th approach can be estimated as follows:

$$y_{i\_est} = \frac{V_{i\_est}}{s_i} = \frac{g_{sat}}{(r + g_{sat})} \quad (17)$$



where:

$y_{i\_est}$	=	estimated flow ratios on the $i$ th approach;
$V_{i\_est}$	=	estimated arrival volumes on the $i$ th approach;
$s_i$	=	saturation flow rate of lane group on $i$ th approach;
$r_i$	=	effective red time; and
$g_{sati}$	=	saturated portion of the green time.

The set of estimated flow ratios are used to determine the sum of critical flow ratios,  $Y$ , from which the minimum delay cycle length can be estimated from Equation 13, given the total lost time per cycle,  $L$ . The base design was used along with some changes to obtain average values for occupancy, saturated green time, green time and cycle length in NETSIM. The minimum green time was set at 1 second (minimum allowed) and the stopline detectors were coded to extend the phase (to maximum) as long as queue was discharging on the approach. Two different vehicle extension values of 1.1 seconds (minimum allowed) and 5.0 seconds were coded. FORTRAN programs were written to reduce the NETSIM detector occupancy information and signal indication on a second-by-second basis to obtain queue clearance time (the saturated portion of the green time), green time, and cycle length by movement and approach. The saturated portion of the green time, the average cycle length, and the saturation flow rate were used to estimate the expected volumes on the approaches by lane group.

The estimated demand volumes would be higher because the lane utilization factor on multi-lane approaches are greater than 1.0. The imbalance in the lane utilization causes the detector to stay on longer and hence accounts for a longer saturated green time. This higher saturated green time translates into a higher estimate of the volumes on the approach. The lane utilization factors can be estimated using Equation 15 from the previous section and used to convert the estimated volumes to get estimates of average arrival volumes on the approach.

## PERFORMANCE MEASURE STUDY

Three different performance measures were considered in this analysis. The performance measures were the total delay per vehicle, the percentage stops and the fuel consumption. These measures were observed for a wide range of scenarios for comparison purposes. In all the scenarios, five different vehicle extension values were used: 1.1, 2.0, 3.0, 4.0, and 5.0 seconds. In the first scenario, all the arrival demand volumes were assumed to be equal. In the next scenario, three approaches had the same arrival demand volume while the fourth had half the volume. In the final scenario, three approaches had the same volume while the fourth approach had double the volume. Approach volumes between 400 and 1400 vph were coded.

Three different minimum green settings and detector types were also coded. In case 1, the stop line detectors were coded to be standard calling detectors and the minimum green time was set at 10 seconds for the left-turning vehicles and 15 seconds for the through and right-turning vehicles. In case 2, the left-turn bay detectors were coded to call and extend until the queue discharges while the through lane stopline detectors were standard calling detectors. In this case, the minimum green for the left-turning traffic was set to 1 second while the through traffic minimum green was 15 seconds. In the final case, all the stopline detectors were coded to call and extend until the queue discharges with the minimum green times for all the phases set at 1 second.

The ability of the TRAF-NETSIM simulation model to simulate isolated signalized intersections under fully actuated control was used to design experiments to accomplish the objectives of this research. The actuated intersection testbed and scenarios for the lane utilization, detector occupancy and performance measures studies were identified and simulated using NETSIM and their respective output reports were generated for subsequent examination. A summary of the above NETSIM-based experimental design is provided in Table 1.

**Table 1. NETSIM Study Design.**

PARAMETER	VALUE
Number of lanes per approach	Two or three lanes serving through and right turning vehicles and One left-turn bay
Phasing Scheme	Standard NEMA (8-phase)
Yellow + All Red Interval	3 + 1 seconds for all phases
Stopline Detectors	12.2 m in length on all lanes Standard Calling or Call and Extend until queue discharges
Upstream Detectors	1.8 m in length on the through lanes
Vehicle Extensions	1.1, 2.0, 3.0, 4.0, or 5.0 seconds
Maximum Green Times	16 seconds for left-turning traffic 41 seconds for the through traffic
Minimum Green Time	1 second (when stopline detectors call and extend) or 10 seconds for left-turn phase (standard calling detectors) 15 seconds for through phase (standard calling detectors)
Volume Split	15% left-turning, 70% through, 15% right-turning
Demand Volumes	A wide range from 400 to 1500 vph per approach
Volume by Approach	Same on all the approaches, or One approach had half the volume as any other, or One approach had double the volume of all others.



## CHAPTER 4

### EXPERIMENTAL RESULTS

#### OVERVIEW

The results from the lane utilization study, occupancy and green time study, and the performance measures study are presented in this chapter. Output results from the NETSIM replications were reduced into comprehensible tables. Data from the different NETSIM replications were averaged and used as inputs to calibrate the lane utilization model and to predict arrival volumes on the approach. The predicted volumes were then used to estimate minimum delay cycle lengths for each scenario.

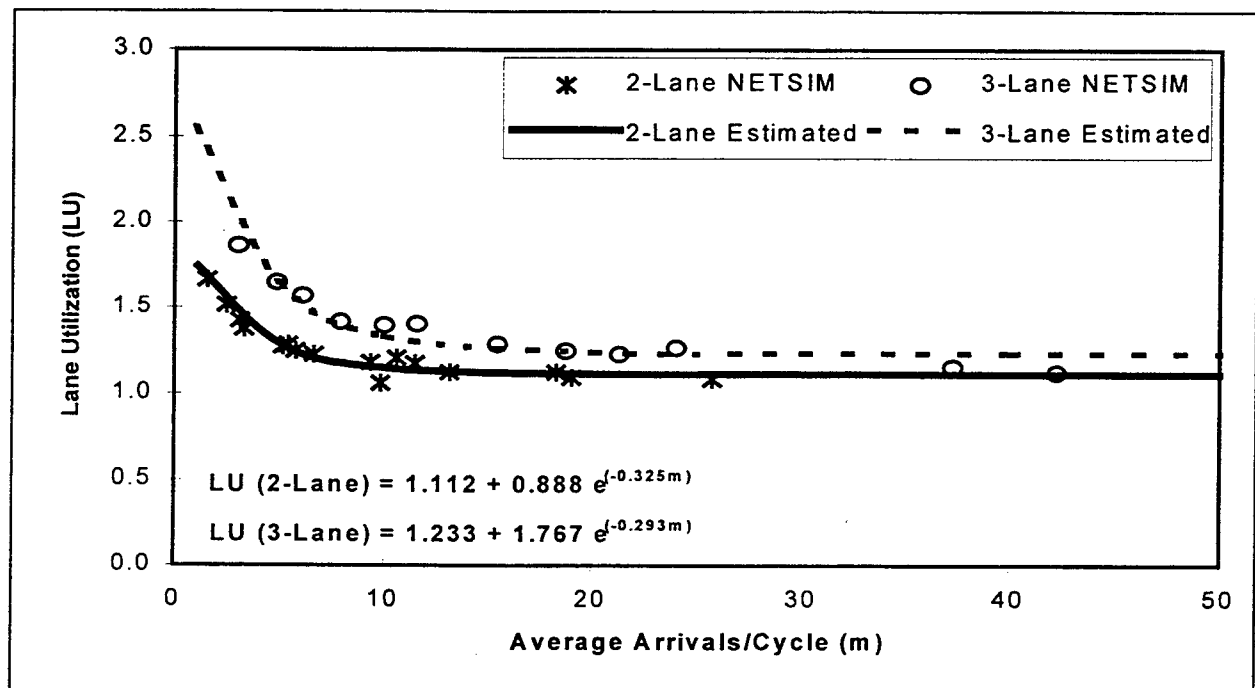
#### LANE UTILIZATION

The simulation runs for the lane utilization study were conducted for two lane and three lane scenarios. Surveillance detector vehicle counts for each lane were obtained on a second-by-second basis. The data were reduced to give the arrivals per cycle by lane and also the average arrivals per cycle for the lane approach using a FORTRAN program. The FORTRAN program reduced the extremely large NETSIM output file to yield arrivals per cycle by lane. The data were then used to estimate NETSIM generated lane utilization values. A plot of these lane utilization values versus the average arrivals per cycle ( $m$ ) showed an exponential decreasing trend with increase in the  $m$  values. The values seem to be asymptotic to a value different from unity. The reduced data was loaded into an Excel spreadsheet and used to calibrate the parameters  $A$  and  $B$ . The 'Solver' function in 'Microsoft Excel' was used to numerically search and obtain parameter values that gave the least squared error between NETSIM observed lane utilization factors and the model estimated factors.

The calibration study results of the lane utilization study are presented in Table 2. Figure 6 provides a comparison of the NETSIM observed lane utilization values and the model estimated lane utilization values for the two and three lane scenarios. From the plots and table it

**Table 2. Calibration of Parameters for Lane Utilization Study.**

No. of Lanes	Parameter A	Parameter B	MSE	R-Square
2	1.112	-0.325	0.001	0.96
3	1.233	-0.293	0.004	0.95



**Figure 6. Model Estimated and NETSIM Lane Utilization Factors.**

is evident that the model estimated lane utilization values are good estimators of NETSIM observed lane utilization values.

### Prediction of Input Volumes

The simulation runs for the occupancy, green time and cycle length study were conducted for two lane scenarios with stopline detectors that called and extended the phase until the queue discharged. Surveillance detector occupancy information for each approach was obtained by

lane group on a second-by-second basis. The data was reduced to obtain the occupancy, green time and cycle length by lane group for each of the four approaches using a FORTRAN program. The program reduced the extremely large NETSIM output file to yield lengths of time the detector is occupied from start of green to the first occasion when the detector is not occupied (representing queue clearance). The occupancy data was then used to obtain the length of the saturated portion of the green time per phase per cycle. The program also reduced the second-by-second signal indication data from the output file to green time for each phase and cycle length values. These values were then averaged over many cycles and NETSIM replications.

It was observed from the reduced data that the average cycle length increased with an increase in approach volume, as expected. The average green time values also showed a similar trend as the average cycle length values. The cycle length for the 1.1 second vehicle extension was lower than the cycle length observed for the 5.0 second vehicle extension cases for all volume conditions. The start up lost time, the average saturated green time, cycle length and red time, along with the saturation flow rate for the through lanes, were then used to obtain an estimate of the arrival volume on the through lanes. The apparent cycle length for the left-turning traffic was higher than that for the through traffic for the following reason. The lower left-turning volumes sometimes leads to no arrivals during a regular cycle (through traffic) causing the phase to skip. A cycle is defined from the start of a red period to the start of the next red period. Consequently, phase skipping was yielding erroneously longer cycle lengths for the left-turning traffic than measured for the through traffic. The saturated portion of green can be used to obtain an estimate of the arrival volumes, but the estimates may not be as reliable as in the case of the higher-volume through traffic.

The estimated arrival volumes were observed to be higher than was input into NETSIM. A major factor that may lead to the over estimation of volumes is the fact that the detector is occupied for a longer time than just based on average values because of an imbalance in lane utilization. The ratio of the estimated volumes to the input volumes were greater than 1.0 for the through lane groups in all the scenarios. The left-turning data yields ratios that are typically

between 0.80 to 1.10, instead of unity.

Table 3 presents the reduced data for one of the scenarios where the approach volume was 800 vph split in the ratio 15% left, 70% through and 15% right turning. Note that the input arrival volume on Approach 1 was 400 vph which is half of 800 vph. The stopline detectors not only called a phase but also extended it until the queue discharged up to the maximum green time for that phase.

**Table 3. Cycle Length, Green Time, Saturated Green Time,  
Estimated Flow Ratio and Volumes by Lane Group.**

		Appr 1		Appr 2		Appr 3		Appr 4	
Veh Ext		Left	Thru	Left	Thru	Left	Thru	Left	Thru
	$V_{actual}$	60	340	120	680	120	680	120	680
1.1 sec	CYC	96.6	64.8	79.7	64.7	70.6	64.8	69.7	64.7
	GRN	5.8	15.4	7.7	19.7	7.4	20.0	7.3	21.5
	Sat GRN	4.8	9.2	6.6	15.0	6.5	14.6	6.4	13.6
	$Y_{i\_est}$	0.030	0.127	0.060	0.224	0.067	0.219	0.065	0.212
	$V_{i\_est}$	53.7	457.1	108.6	806.6	119.9	789.6	117.8	764.2
	$V_{est}/V_{act}$	0.89	1.34	0.90	1.19	1.00	1.16	0.98	1.12
5.0 sec	CYC	120.7	99.6	103.7	99.7	99.7	99.6	100.2	99.7
	GRN	6.1	28.7	9.0	36.0	9.8	35.1	9.6	36.9
	Sat GRN	5.1	12.6	8.2	19.0	8.9	18.7	8.9	19.1
	$Y_{i\_est}$	0.027	0.130	0.061	0.211	0.071	0.205	0.071	0.214
	$V_{i\_est}$	47.7	469.3	110.5	759.3	128.3	738.8	126.9	769.9
	$V_{est}/V_{act}$	0.80	1.38	0.92	1.12	1.07	1.09	1.06	1.13



### Minimum Delay Cycle Length

The critical flow ratios from the above analysis can then be used in the estimation of a minimum delay cycle length for the given scenario. Note that the flow ratio for the left-turning traffic is estimated based on a longer cycle length (due to phase skipping). The critical flow ratios were identified and the actuated intersection minimum delay cycle lengths were calculated using Equation 13. The results of the minimum delay cycle length estimation are presented in Table 4 below.

**Table 4. Minimum Delay Cycle Length Estimates.**

Demand Volume (vph)		Actual Sum Crit. $Y_i$	Cycle Length		
Approach 2, 3, 4	Approach 1		NETSIM Generated		Courage's Min. Delay
			Veh Ext = 1.1	Veh Ext = 5.0	
400	Half	0.256	35.6	57.4	27.9
600	Half	0.383	48.5	79.5	33.7
800	Half	0.511	64.7	99.6	42.5
1000	Half	0.639	84.4	113.3	57.6
500	Double	0.479	55.5	92.9	39.9

Estimated Minimum Delay Cycle Length							
Cycle Length (using predicted volumes)				Cycle Length (accounting for lane utilization)			
Sum Crit $Y_i$	Veh Ext = 1.1	Sum Crit $Y_i$	Veh Ext = 5.0	Sum Crit $Y_i$	Veh Ext = 1.1	Sum Crit $Y_i$	Veh Ext = 5.0
0.331	31.1	0.323	30.7	0.257	28.0	0.270	28.5
0.465	38.8	0.451	37.9	0.404	34.9	0.410	35.3
0.575	49.0	0.561	47.4	0.525	43.8	0.518	43.2
0.674	63.9	0.675	64.0	0.622	55.0	0.623	55.2
0.541	45.4	0.512	42.6	0.484	40.3	0.469	39.1

The minimum delay cycle lengths estimated using predicted volumes were slightly higher than those predicted using actual demand volumes. However, the results were much closer after the predicted volumes were adjusted to account for unbalanced lane utilization.

## **TRAFFIC PERFORMANCE MEASURES**

Three different traffic performance measures were observed to identify trends in the data sets. It was observed that the delay, percentage stops and fuel consumption increased as volume increased. From the different scenarios, it was also observed that the average intersection delay per vehicle in most cases increased with increasing vehicle extension values. The stops showed a trend that is opposite to that observed in average total delay. The trends in fuel consumption weren't significant as the differences in the fuel consumption values were very small.

### **Average Intersection Delay**

Delays were averaged over each of the four approaches and NETSIM replications. Some trends that were observed are presented below. Figures 7, 8, 9 and 10 present the average intersection delay per vehicle versus the vehicle extension values for the different volume levels and different minimum green time and stopline detector configurations. In the scenario where the stopline detectors not only call the phase but also extend it until the queue discharges, it can be seen from Figures 8 and 9 that as the volumes increased, the delay increased, and as the vehicle extension value increased the delay values also increased up to a certain maximum value.

In the other two scenarios presented in Figures 9 and 10, where the stopline detectors were coded as standard calling detectors with a minimum green time of 10 seconds for the left phases and 15 seconds for the through phases, delay increased with an increase in volume. For the 1200 vph volume condition, the delay values first decreased with an increase in the vehicle extension value and then the delay increased as the vehicle extension value was further increased. In the lower extension case, premature gap out in the presence of queues lead to the increased delays. The effect lasted until a critical vehicle extension value was reached beyond which the increased green time along with increased cycle length values led to longer delays. This trend

was also observed in the volume condition where one approach had half the volume on the other approaches for a volume of 1400 vph. It is possible to obtain an optimal vehicle extension time and the optimal cycle length (based on minimum delay) for this particular case.

### **Percentage Stops**

The percentage stops observed were averaged over each of the four approaches and NETSIM replications. Some trends that were observed are presented below. Figures 11, 12, 13 and 14 present the percentage stops versus the vehicle extension values for the different volume levels and different minimum green and stopline detector configurations. A reverse trend is observed in the case of percentage stops, i.e. the percentage stops decrease with an increase in the vehicle extension values. At the lower extension values, the green time ends shortly after the queue discharges and very few vehicles go through the intersection without stopping.

The first two plots (Figure 11 and 12) present the percentage stops versus vehicle extensions for detector settings where the stopline detectors extend the phase until queue clears. It was observed that as the volumes increased the number of stops increased. From the plots it is noted that as the vehicle extension values increased the percentage of stops decreased. This trend is not so evident for the lower volume cases where the percentage stops are almost a constant for all vehicle extensions. The next two plots (Figure 13 and 14) present similar graphs for the case where the minimum green time is set to 10 seconds for the left-turning phases and 15 seconds for the through phases and the stopline detectors call the phase but do not extend it. Similar trends were observed in percentage stops varying with approach volumes and vehicle extension values.

### **Fuel Consumption**

The fuel consumption observations were averaged over each of the four approaches and NETSIM replications. No significant trends were observed other than fuel consumption increased as the approach volumes increased. Figures 15, 16, 17 and 18 present the fuel consumption in gallons versus the controller vehicle extension values for the different volume levels and different minimum green and stopline detector configurations.

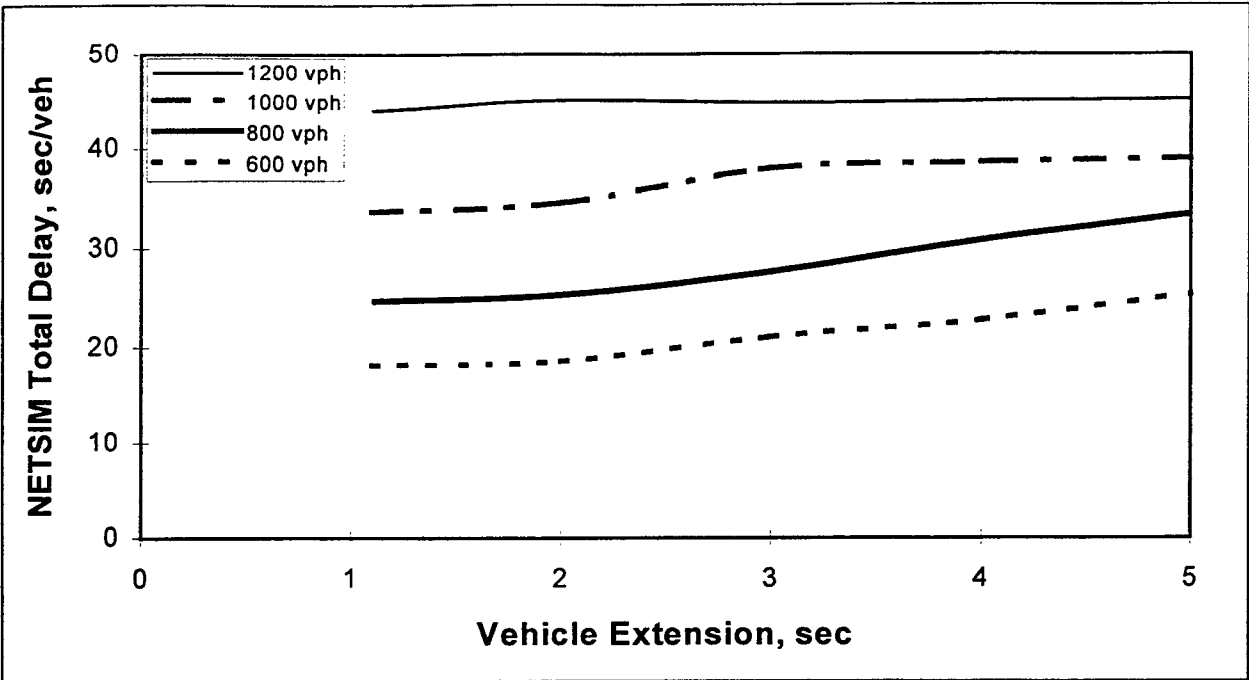


Figure 7. Total delay versus Vehicle Extension for Queue Clearance Scenario when All Approaches Have the Same Volume.

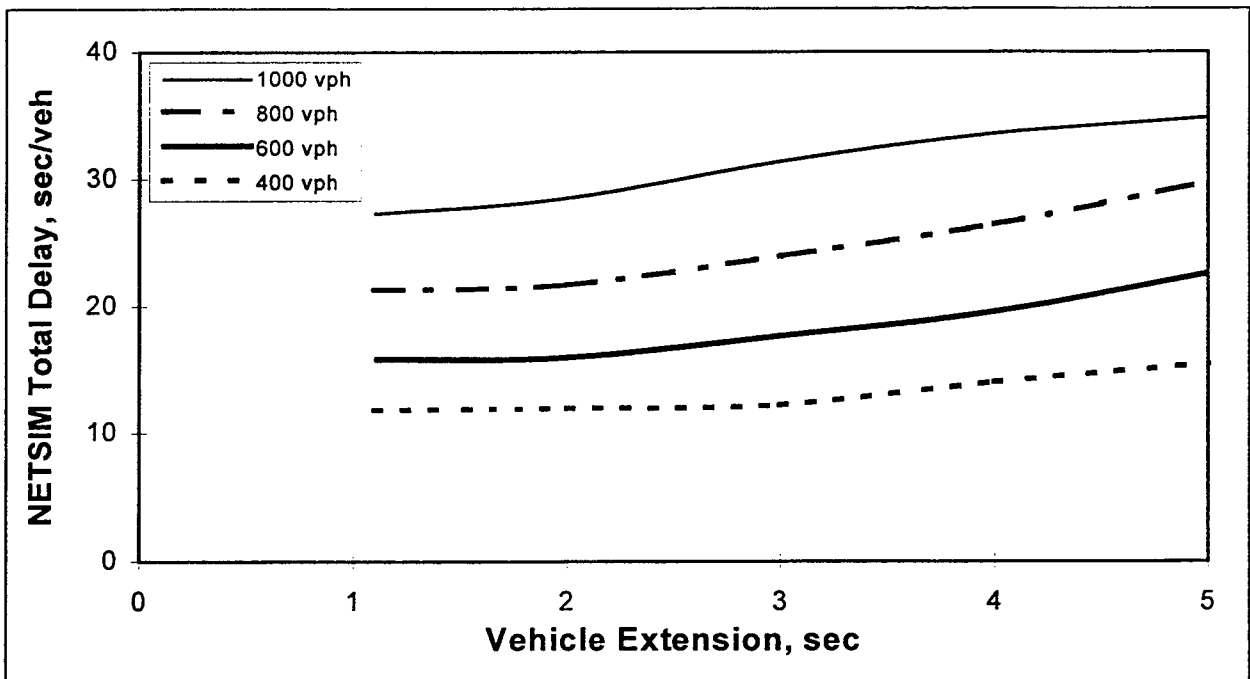


Figure 8. Total Delay versus Vehicle Extension for Queue Clearance Scenario when Approach 1 Has Half the Volume of the Other Approaches.

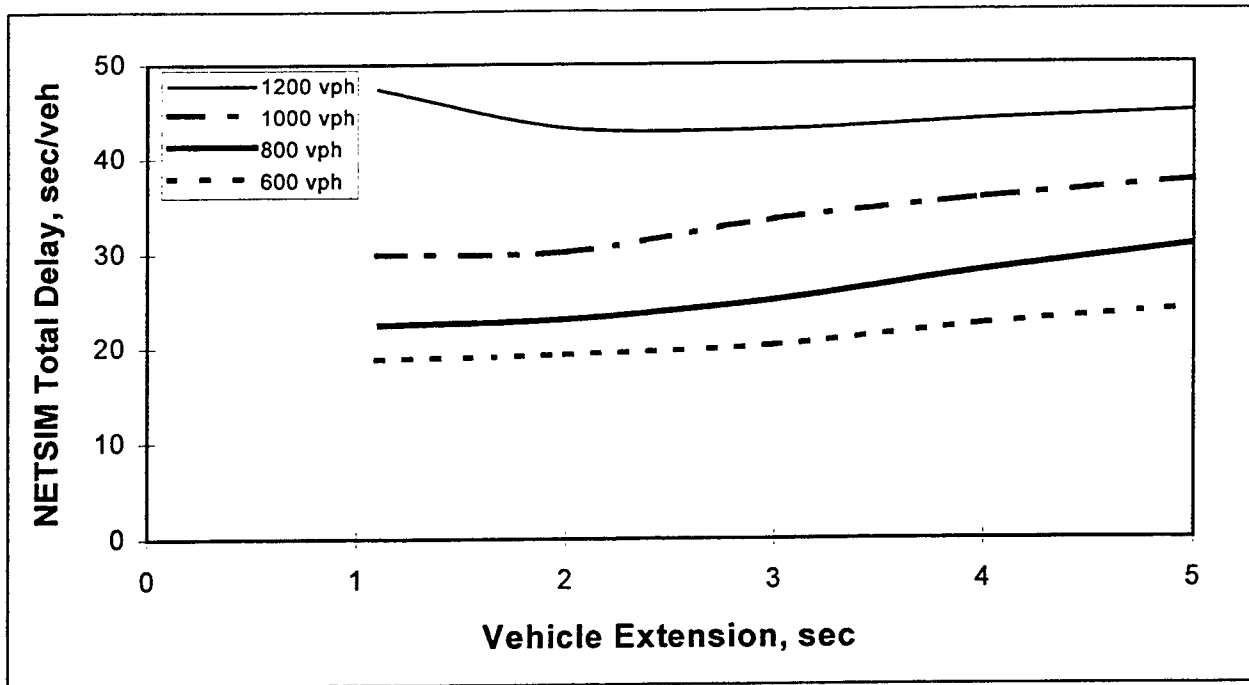


Figure 9. Total Delay versus Vehicle Extension for Minimum Green Scenario when All Approaches Have the Same Volume.

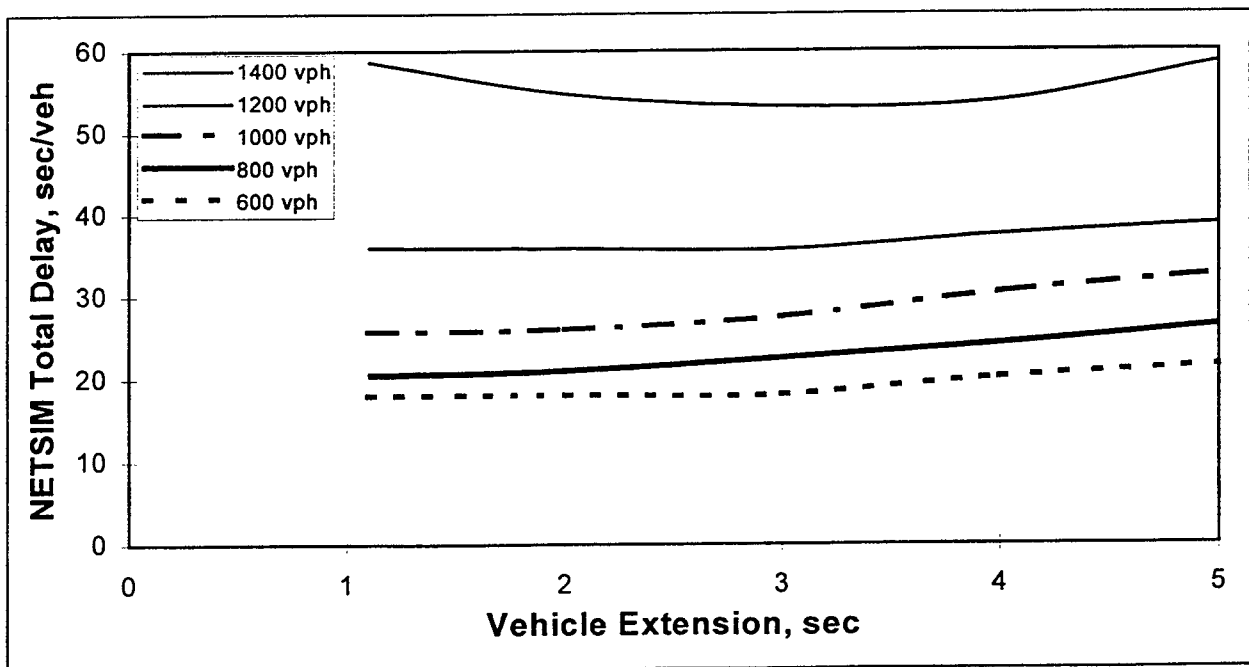
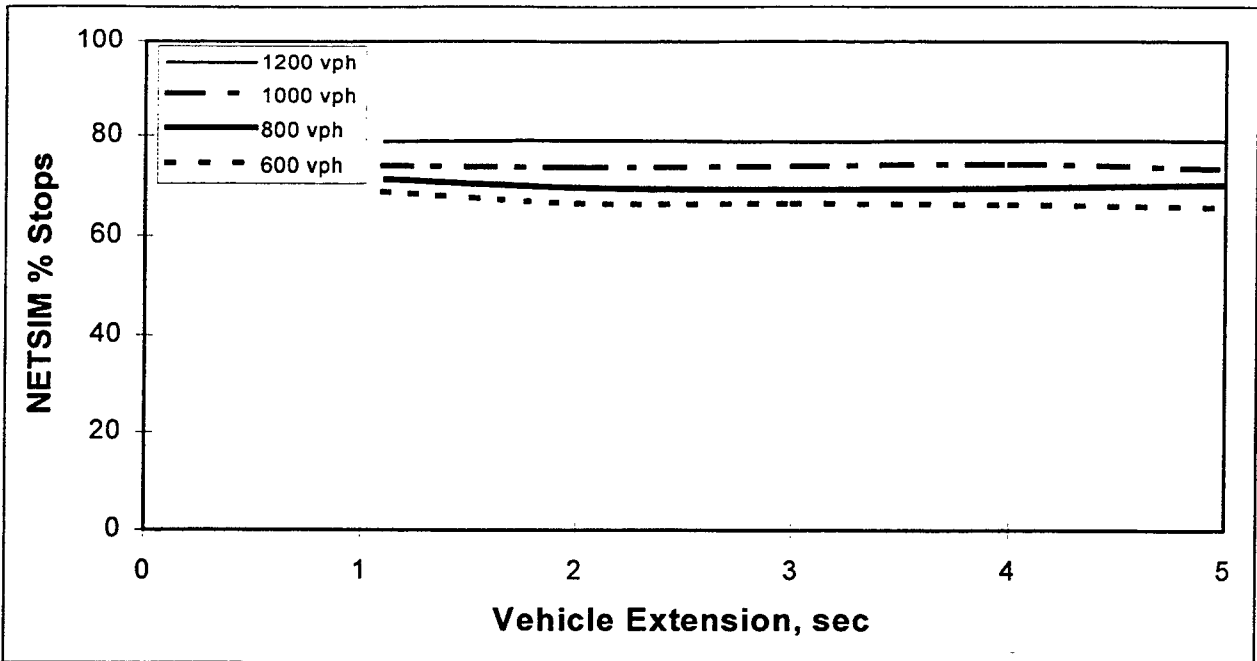
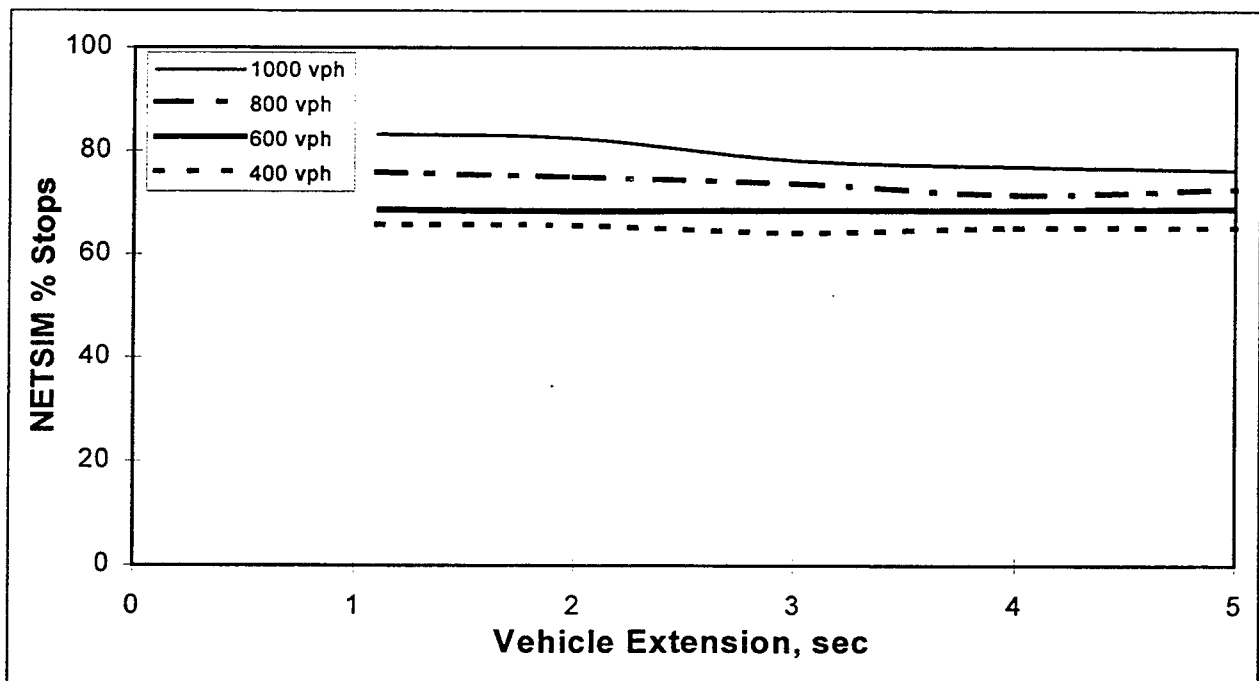


Figure 10. Total Delay versus Vehicle Extension for Minimum Green Clearance Scenario when Approach 1 Has Half the Volume of the Other Approaches.



**Figure 11. Percentage Stops versus Vehicle Extension for Queue Clearance Scenario when Approaches Have the Same Volume.**



**Figure 12. Percentage Stops versus Vehicle Extension for Queue Clearance Scenario when Approach 1 Has Half the Volume of the Other Approaches.**

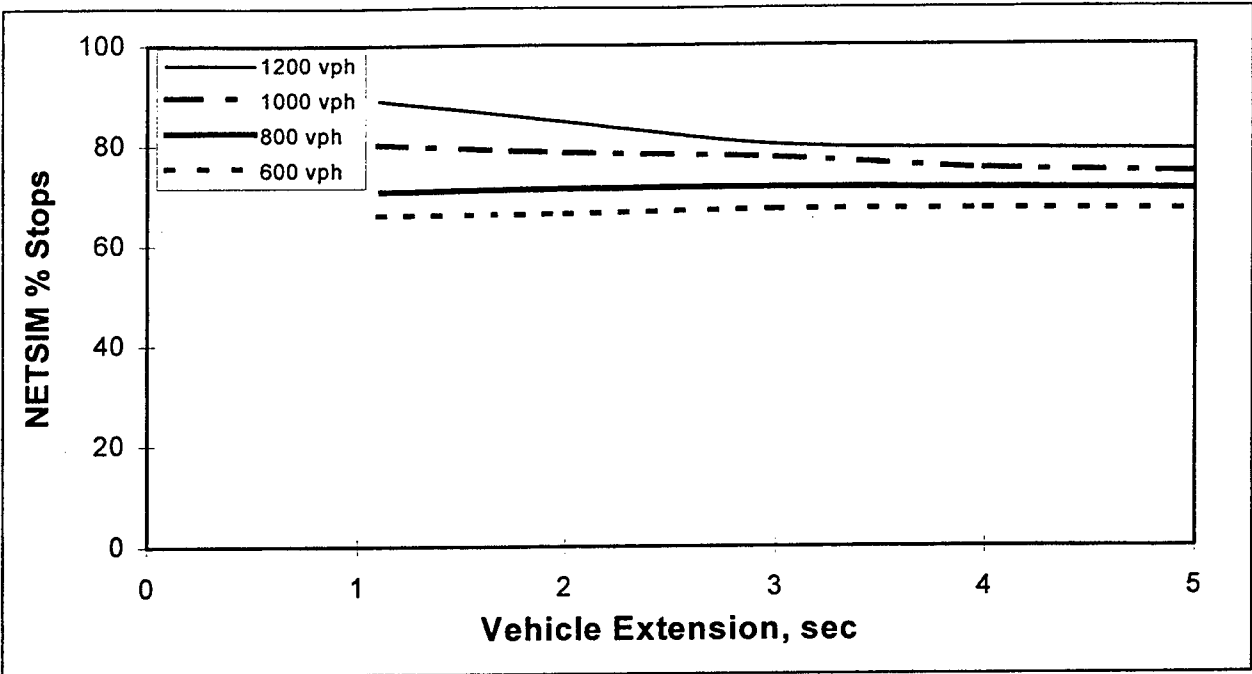


Figure 13. Percentage Stops versus Vehicle Extension for Minimum Green Scenario when all Approaches Have the Same Volume.

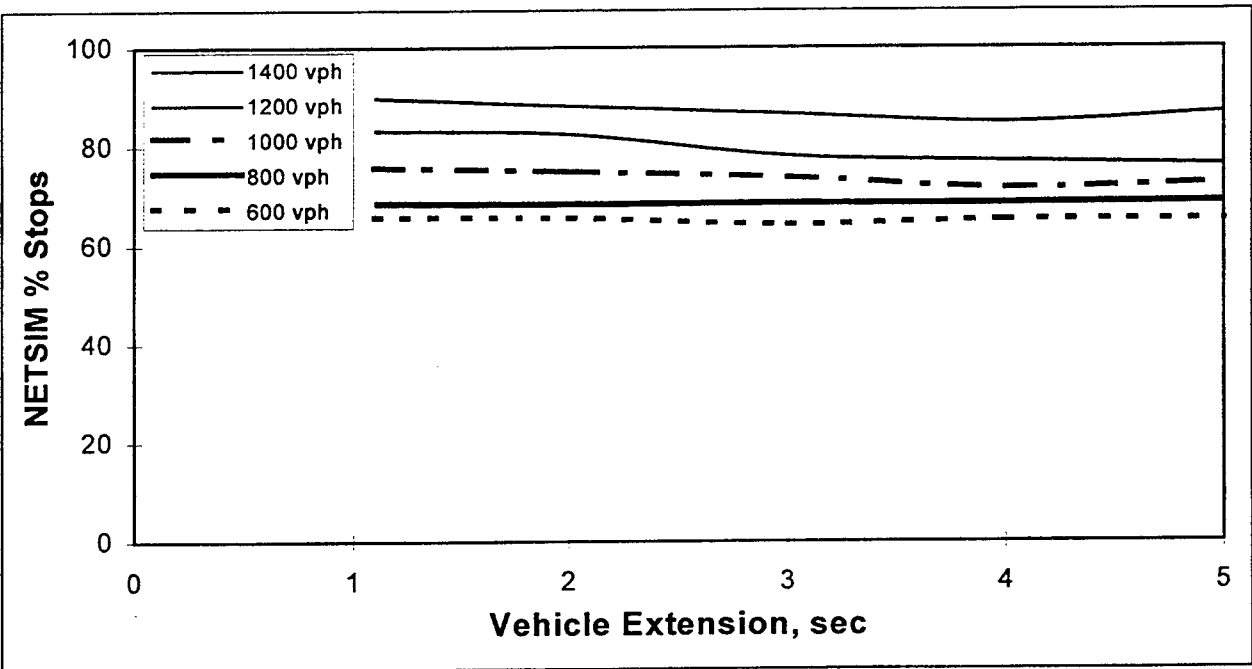
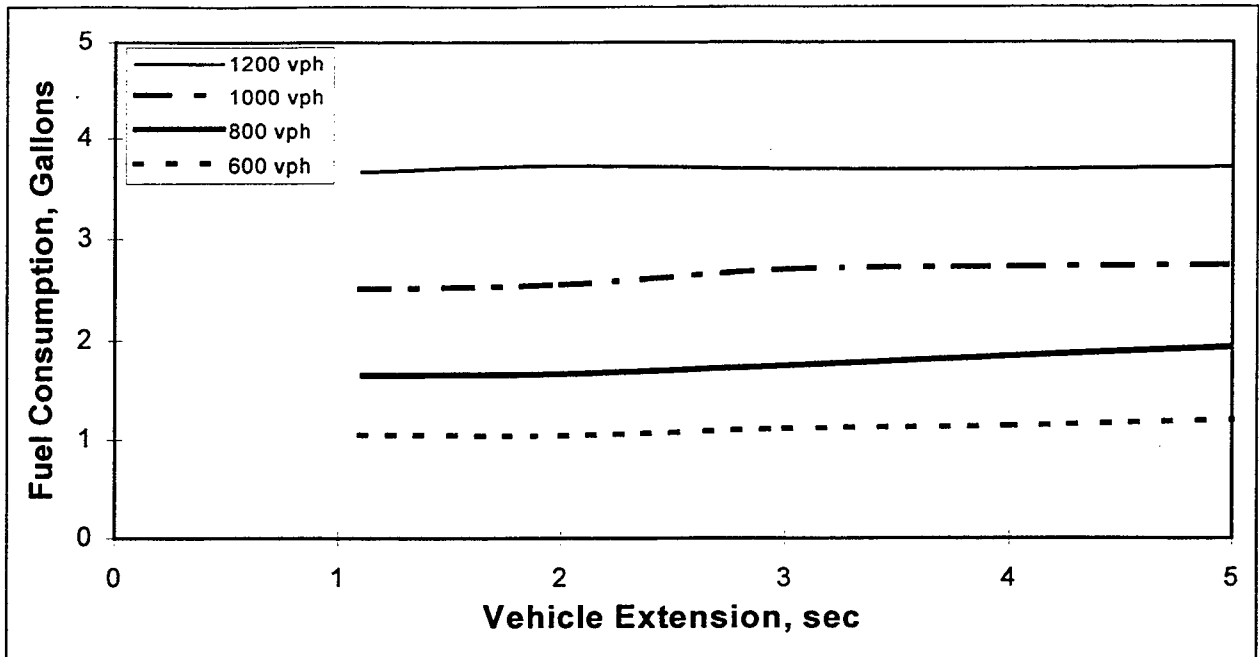
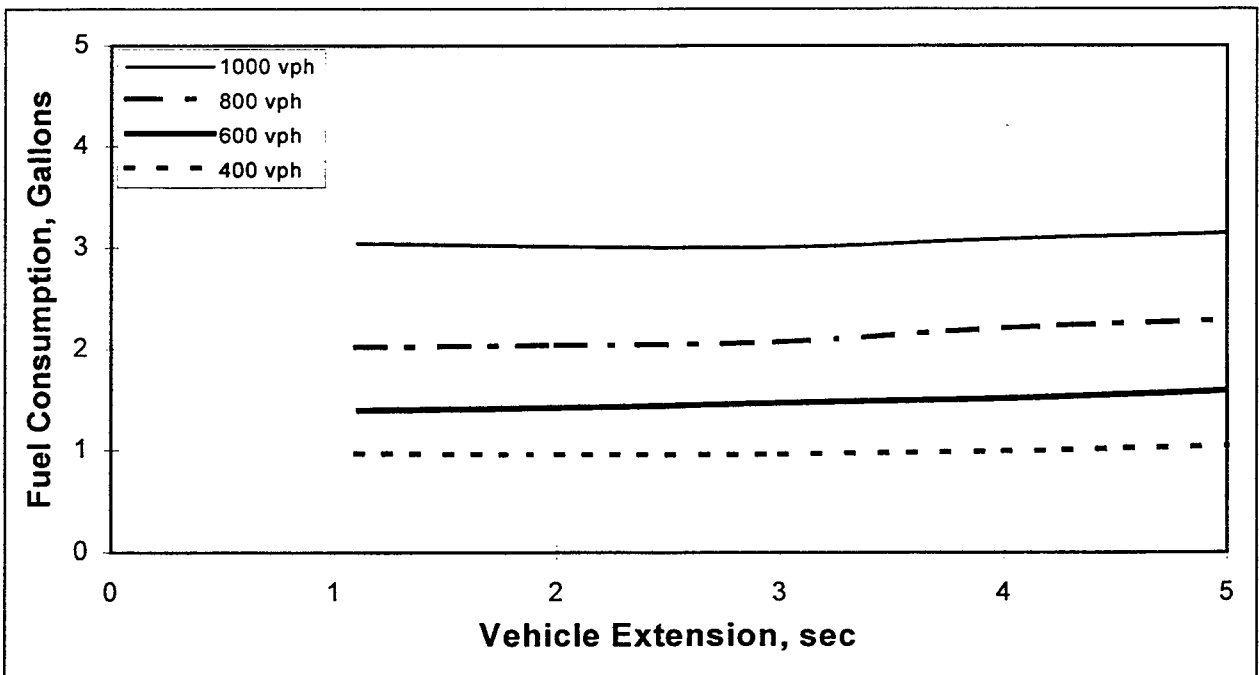


Figure 14. Percentage Stops versus Vehicle Extension for Minimum Green Clearance Scenario when Approach 1 Has Half the Volume of the Other Approaches.



**Figure 15. Fuel Consumption versus Vehicle Extension for Queue Clearance Scenario and All Approaches Have the Same Volume.**



**Figure 16. Fuel Consumption versus Vehicle Extension for Queue Clearance Scenario when Approach 1 Has Half the Volume of the Other Approaches.**



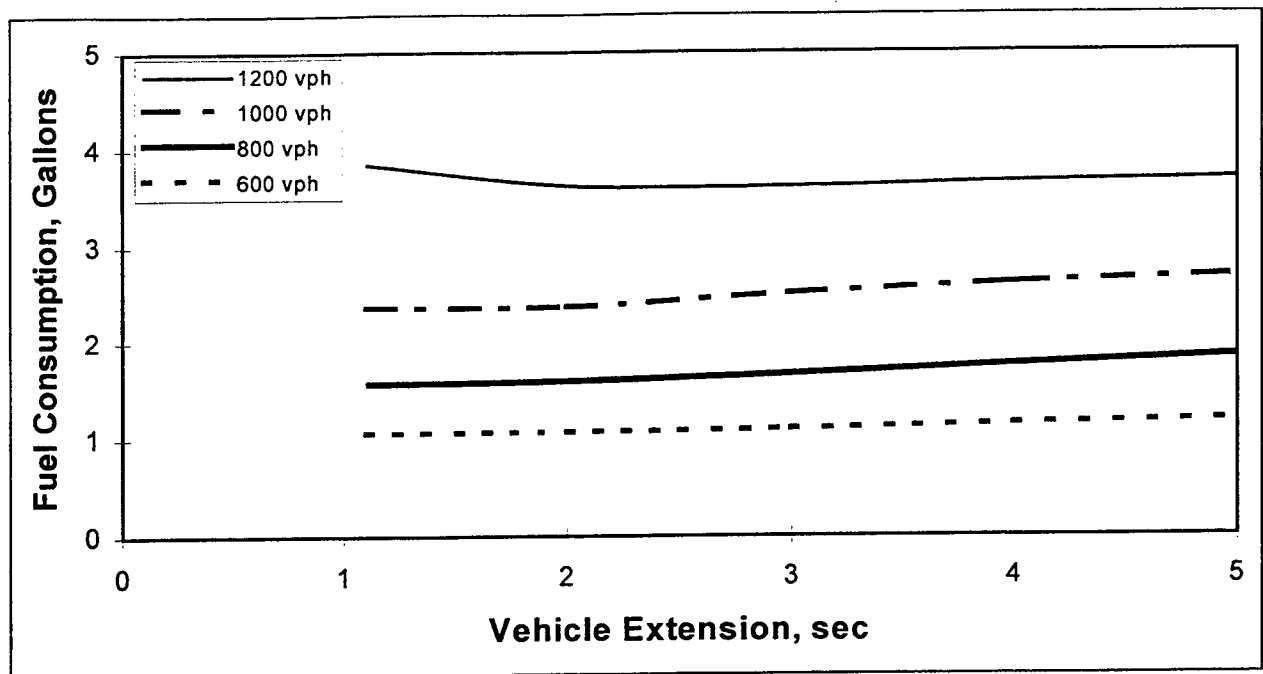


Figure 17. Fuel Consumption versus Vehicle Extension for Minimum Green Scenario when All Approaches Have the Same Volume.

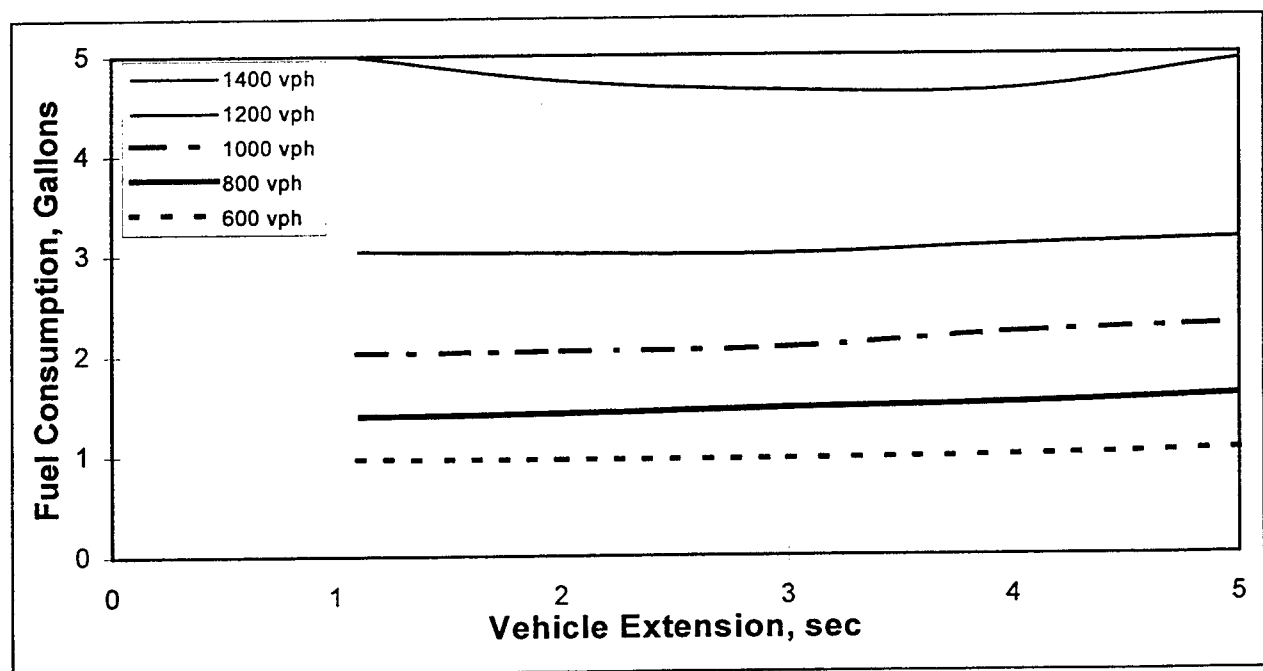


Figure 18. Fuel Consumption versus Vehicle Extension for Minimum Green Clearance Scenario when Approach 1 Has Half the Volume of the Other Approaches.

## **MINIMUM GREEN**

Some concluding simulation studies were conducted using the TRAF-NETSIM traffic simulation software to examine the relative sensitivity of several of the principal parameters of actuated signal control. These parameters include the Minimum Green selected, the type of traffic detector utilized (stopline and arrival), and mode of operation (pulse and presence). The following simulation results were observed for the isolated intersection having traffic actuated control.

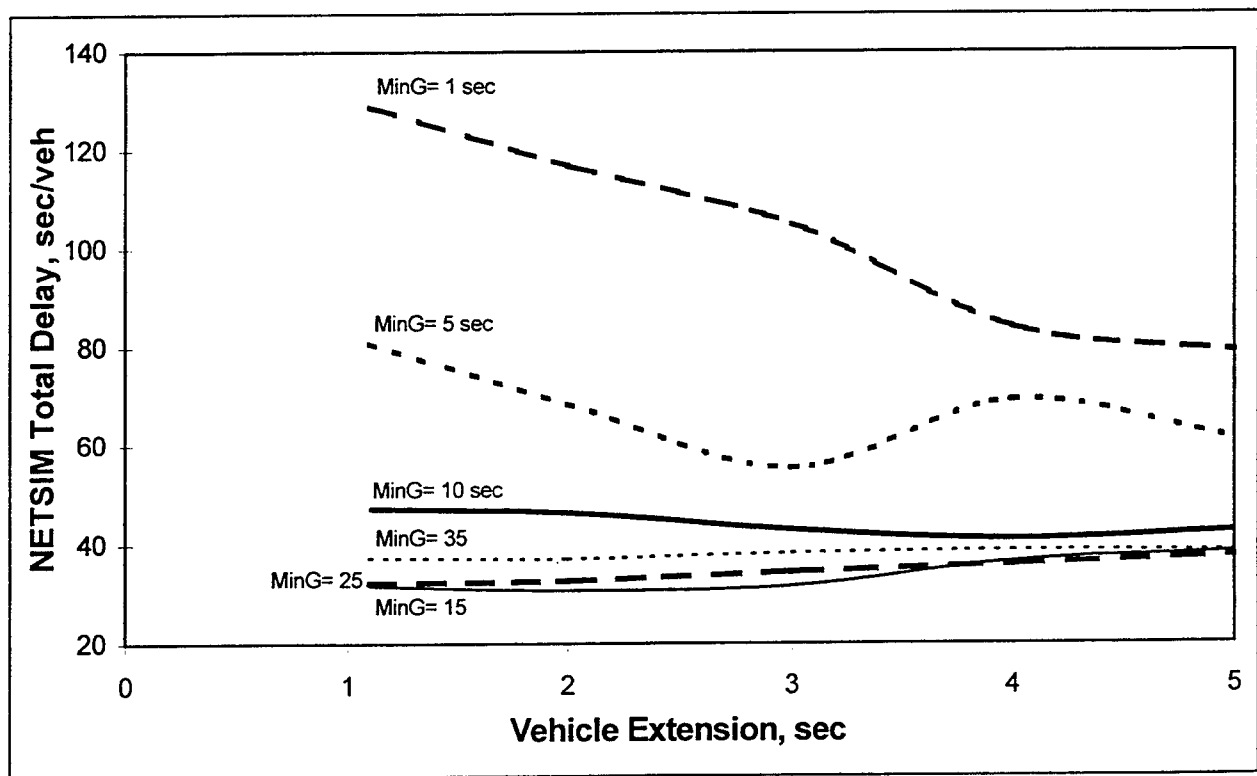
### **Study Design**

The base design was used along with some changes to study the impact of minimum green time on average intersection delay. The minimum green times for the through phases were varied from 1 second (minimum allowed) to 35 seconds, and the stopline detectors were coded as standard calling only detectors. The left turn bays had 12.2 meters long stop line detectors that were coded to call and extend until the left turning queue disappeared or the maximum green time for the left turn phase was reached, whichever occurred first. The upstream advance detectors on the through lanes were 1.8 meters long and coded to operate in the pulse mode and were allowed to call and extend the phase. Five different vehicle gap extension values of 1.1, 2.0, 3.0, 4.0 and 5.0 seconds were coded. A fairly high arrival volume of 1000 vph was coded on all four approaches. Approach volumes were split 15%, 70% and 15% for the left-turning, through and right turning traffic. Simulation runs were replicated ten times with different random seed numbers and the results obtained were averaged over them.

### **Study Results**

The intersection delay reported were averages over all replications and movements. Figure 19 provides a plot of vehicle extension versus intersection delay for different minimum green time settings. It can be observed from the plot that the intersection delay was very high when the minimum green time was very low. In the low minimum green value cases, premature gap out occurred and a queue remained at the end of green in a cycle. Premature gap out may occur due to the fact that the vehicle extension value is low or due the possibility that a queued

vehicle is stopped over the upstream call and extend detectors which is in the pulse mode. In effect, the capacity of the phase is low and overflow occurs during every cycle. The throughput for the 1 and 5 second minimum green time cases was observed to be lower than the arrival demand. The intersection delay approached minimum values for a minimum green time of 15 seconds for this volume level. The delay values did not vary much for minimum green time values from 15 to 25 seconds. Beyond a 25 second minimum green time, the intersection delay values increased again with an increase in minimum green time.



**Figure 19. Variation in Total Delay with Minimum Green versus Vehicle Extension.**

## **DETECTOR TYPE**

### **Study Design**

The base design was used along with some changes to study the impact of detector type on average intersection delay. One scenario was simulated with a minimum green time of 1 second along with stopline detectors that were coded as standard calling only detectors. The other scenario also used a minimum green time of 1 second along with stop line detectors that were coded to call and extend until the queue discharges or the maximum green time for the through phase is reached, whichever occurred first. The upstream advance detectors on the through lanes were 0.3 meters long and two modes of operation (pulse and presence) were simulated. The upstream detectors were allowed to call and extend the phase. Five different vehicle extension values of 1.1, 2.0, 3.0, 4.0 and 5.0 seconds were coded. The left turn bays had 12.2 meters long stop line detectors that were coded to call and extend until the left turning queue disappeared or the maximum green time for the left turn phase was reached, whichever occurred first. An arrival volume of 1000 vph was coded on all four approaches. Approach volumes were split 15%, 70% and 15% for the left-turning, through and right turning traffic. Simulation runs were replicated ten times and the results obtained were averaged over them.

### **Study Results**

The delay values were averaged over replications and over the different movements and approaches. Figure 20 provides a plot of vehicle extension versus intersection delay for different detector type settings. It can be observed from the plot that the intersection delay is very high for the scenario in which the stop line detectors are standard calling detectors for all vehicle extension values. The low minimum green time value of 1 second causes the phase to prematurely gap out and a queue remains at the end of green in a cycle. Premature gap out may occur due to the fact that the vehicle extension value is low or due the possibility that a queued vehicle is stopped over the upstream call and extend detectors which is in pulse mode. In effect the capacity of the phase is low and overflow occurs during every cycle. When the upstream detectors are coded to operate in the presence mode, the intersection delay values drop to much lower values. In the low vehicle extension case, there is still premature gap out leading to higher

delays and lower throughput.

From Figure 20 it can be observed that for the scenario in which the stop line detectors were coded to call and extend until the queue discharges, the mode of operation (pulse or presence) of the upstream detectors didn't have an impact on intersection delays. The delay values were lower than for the case where the stop line detectors were standard calling detectors.

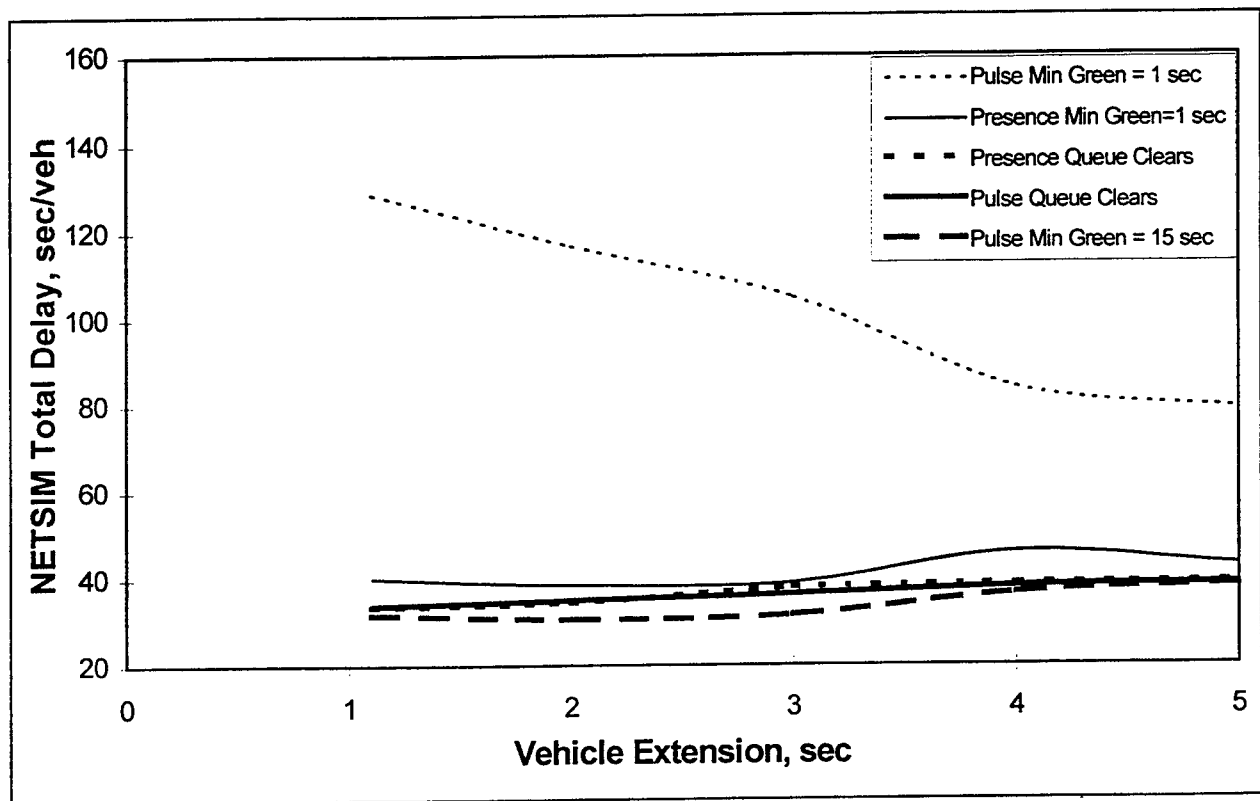


Figure 20. Variation in Total delay with Detector Mode versus Vehicle Extension.



## **CHAPTER 5**

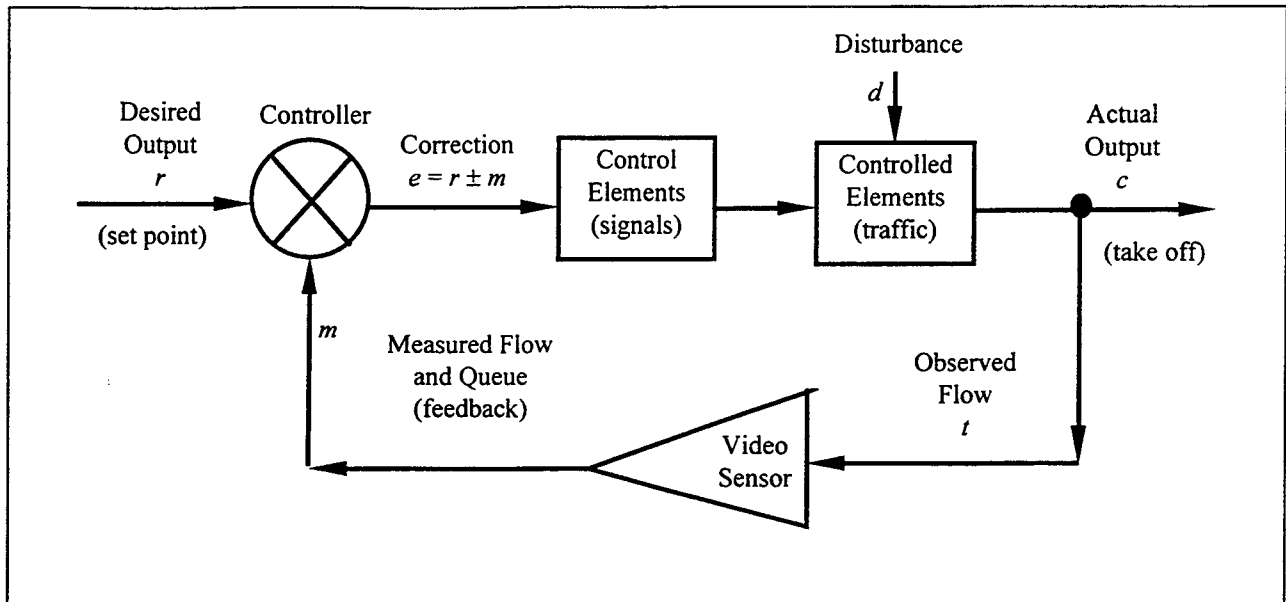
### **IMPLEMENTATION**

#### **INTRODUCTION**

This research followed a proposed experimental plan to completion with the full expectation that only one traffic actuated signal control system design would be recommended—the optimal one. After closely examining our results, we believe that three system designs should be presented and discussed. All appear to lead, practically speaking, to similar optimal control results, but in different operational ways. Two designs provide formal process control system design structures. The first of these requires complex and difficult direct measurement of the controlled traffic process, e.g., traffic queuing. The second uses indirect measurement methods to estimate queuing. The third design concept, which has evolved from much field experience and is used today by some technically expert traffic engineers, appears to be a near optimal design mutation. It measures queues indirectly in real time and has design features that provide a near optimal local control process with little need to redesign present NEMA TS 2 controllers. With this latter control system, the initial traffic signal design and operating strategy must be optimal, however. One problem with this latter design remains that its optimal performance currently is not verifiable in the field with current monitoring strategies.

#### **DIRECT FEEDBACK CONTROL SYSTEM**

Conceptually, a traffic actuated signal controller should be designed to provide the salient features of feedback control systems presented in Figure 21. Of particular importance are the two features of (1) having a real-time objective function (or single “set-point” value) of the control process and (2) having real-time feedback obtained from measurements of the traffic control process itself. The real-time objective function should be defined for (or by) the local controller in traffic flow units that it can measure at the intersection. Even the initial research notion, having a defined optimal cycle time for a given measured traffic volume level, is one step removed from the ideal “set point” measurement system of measuring the flow process in the same units that the controller’s objective function is seeking to optimize (e.g., delay, stops, etc.).



**Figure 21. Recommended Basic Block Diagram and Information Flow of a Traffic Actuated Controller Unit Having Information Feedback for Correcting Actual Output.**

The assessment of reality for the recommended system design presented in Figure 21 is that it probably cannot be readily provided today by economically viable systems, even with ITS technology-level video imaging system capabilities. Vehicular queue counting and signal delay measurement by automated systems are extremely difficult to make with satisfactory precision and reliability. However, should these capabilities become feasible in the future, then the system design proposed in Figure 21 would suddenly become a viable design option.

### INDIRECT FEEDBACK CONTROL SYSTEM

The feedback in this actuated control system is slightly different from that described above. For this system, the feedback is an indirect measure of cycle time,  $C(t)$ , and phase utilization,  $g_p(t)$ , rather than a direct measure of the traffic flow process to be optimized, i.e., queuing delay (and more difficult to measure in the field). In this indirect system, traffic



volumes, per se, and delays would not be directly observed or counted on site, but they would rather be inferred from indirect measurements and traffic flow theory. Only on/off times of discrete process events are noted. Figure 22 provides an overview of the proposed control system optimization algorithm. For example, REDZRO and GRNZRO are the real-time starting times of the red and subsequent green signal intervals for a phase, and QUEOFF is the real-time measured end of vehicular presence of the phase's stopline queue detector set. The basic theory of this control system was described in Chapter 2. The general control system design, however, also follows the information feedback depicted in Figure 21 as well. The NETSIM simulation studies of Chapter 3 suggest that this method would provide very good results.

The optimal “steering function” for this control system design (Figure 22) would adjust the size of the primary signal control variables (Min GRN and Passage GAP) in such a way as to steer control toward the “optimal cycle length” for the intersection. The simulation studies, traffic theory, and field experience all indicate, for basic actuated control without stopline detectors, that the average operating cycle length provided by the controller is a function of Min GRN and Passage GAP set in the controller for each phase. Shorter cycles result from shorter Min GRN and Passage GAP times. It appears from Figure 19 that Min GRN is the more sensitive variable in causing delay if the upstream arrival detectors (for phase extension) operate in the pulse mode, due to premature phase gap-out, but Min GRN is less sensitive to delay causation when the upstream arrival detectors are in the presence mode (See Figure 20.).

Clearly, a signal design should be avoided that has very short fixed minimum green times with no stopline detectors together with its arrival detectors set for pulse mode of operation. For intersection signal designs that provide stopline queue detection, the Min GRN adjustments shown in Figure 22 are unnecessary. Only Passage Gap adjustments would be needed.

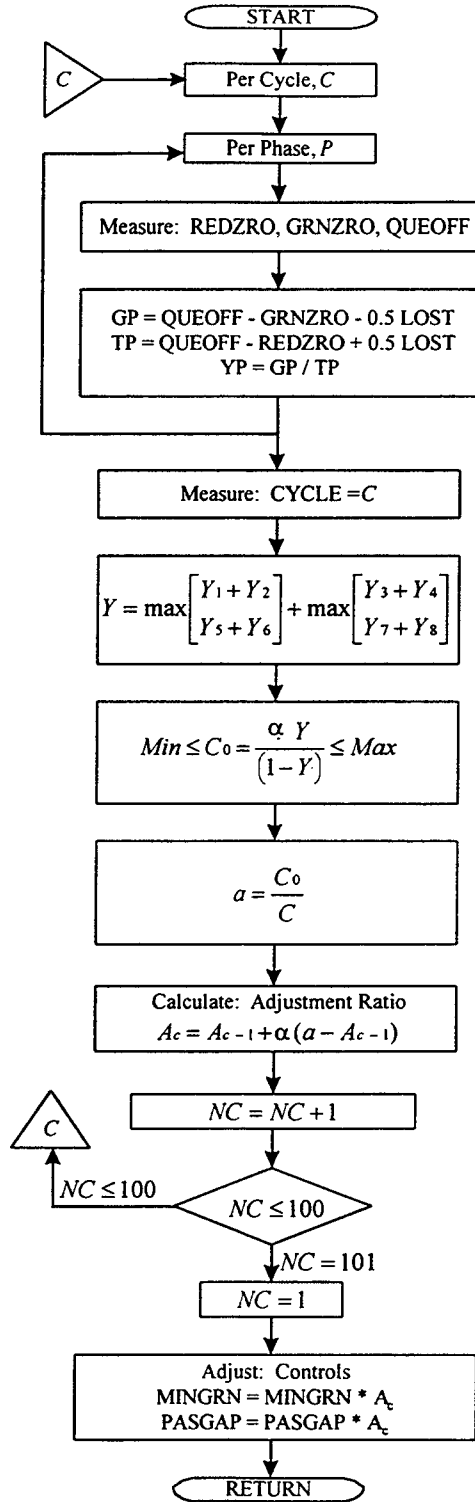


Figure 22. Optimal Cycle-based Feedback Control System.

## NEMA TS2

This research has shown that near optimal traffic actuated signal operation to minimize delay or fuel consumption can be provided with a modern NEMA TS2 traffic actuated controller provided that the original intersection signal design plan and controller parameter selection are optimized. The real-time measurement of arrival flows on the approach and queues at the stopline of the intersection on a cycle-by-cycle basis eliminates errors in modeling assumptions and mathematical expressions needed for more formal optimization algorithms. Signal control green splits and cycle lengths are produced that respond in proportion to the cyclic variations in measured traffic demand. Enhanced actuated systems should verify that the green split proportions being provided in real-time are in fact in proportion to the arrival flow rates.

The principal operational problem in maintaining a near optimal cycle length may result from using multiple loop approach detection systems that are sometimes used to address “dilemma zone” problems associated with phase termination. Future multiple-loop signal designs should contain self-organizing functions that can determine which loops should be monitored to optimize traffic operations for existing conditions.

The following design process is recommended. First, the signal phase selection should minimize the number of protected (exclusive), conflicting phases. Protected phases should be used only where operational capacity, legal requirements, or traffic safety needs dictate. This act would minimize the cycle length operating at any volume level, and minimize the total lost time in the system. Second, given that the minimum number of required protected conflicting phases is established, then the number of concurrently timed and/or overlap protected phases should be maximized, subject to pedestrian crossing considerations. Third, stopline calling and queue extension detection (Extended Call/Delayed Call) should be employed on each lane at the stopline for all phases. These detectors should be inhibited (turned off) when their queue presence plus preset extension time (or equivalent threshold saturation flow headway) is first lost following green onset. The functional purpose is to identify to the controller when the stopline saturation flow has ended because the approach queue has been fully serviced by the current

green signal. Only (random) arrival traffic remains to be serviced by the phase. The ideal design would provide one stopline detector for each lane, each with its own detector (channel). The Min GRN (minimum green) times should be set in the controller as low as possible subject to driver expectancy considerations for phase duration. It should not be necessary to calculate minimum green times to satisfy stopline queues, as the phase should not end until the existing maximum queue per lane is serviced each cycle. Lane utilization issues are automatically addressed by the detection system. The controller feature of “Variable Minimum Green”, provided by existing NEMA controllers since the days of “volume-density” control, should not be used as an inexpensive substitute in this design for the recommended stopline detection plan.

Should multiple-loop approach detection be installed, excessive green extension may occur per phase and either PasGAP reduction techniques using Figure 22 may be applied or detector station reduction techniques may be desired to reduce the operating cycle to minimum delay values for the given volume level.

One operational feature that might be examined in the future contains behavioral modification implications of drivers to achieve more efficient traffic control. One might operate a traffic actuated phase serving a multiple-lane approach having separate detectors on each lane in a radically different manner than is currently done. In this design, each lane would have its own Gap-Out function, rather than having all detector (lane) inputs being combined into one phase as is common practice. During high-volume conditions, the current phase would gap-out when the first lane gaps out, implying an inefficient use of the remaining phase capacity. Motorists would become aware of this operation either from experience and/or special signing, and presumably they would try to maintain a balanced lane utilization thereby keeping the green as long as possible (needed). Changeable lane assignments signing systems could also simultaneously operate at the signal to further balance lane utilization and assist in the optimal assignment of approach capacity to variable turning movement volumes. The safety aspects of this type of system should be thoroughly investigated, however.

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